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Environmental Impact Research Program

Hydraulic Impacts of Riparian Vegetation; Summary of the Literature

by J. Craig Fischenich

WES

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Prepared for Headquarters, U.S. Army Corps of Engineers

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U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

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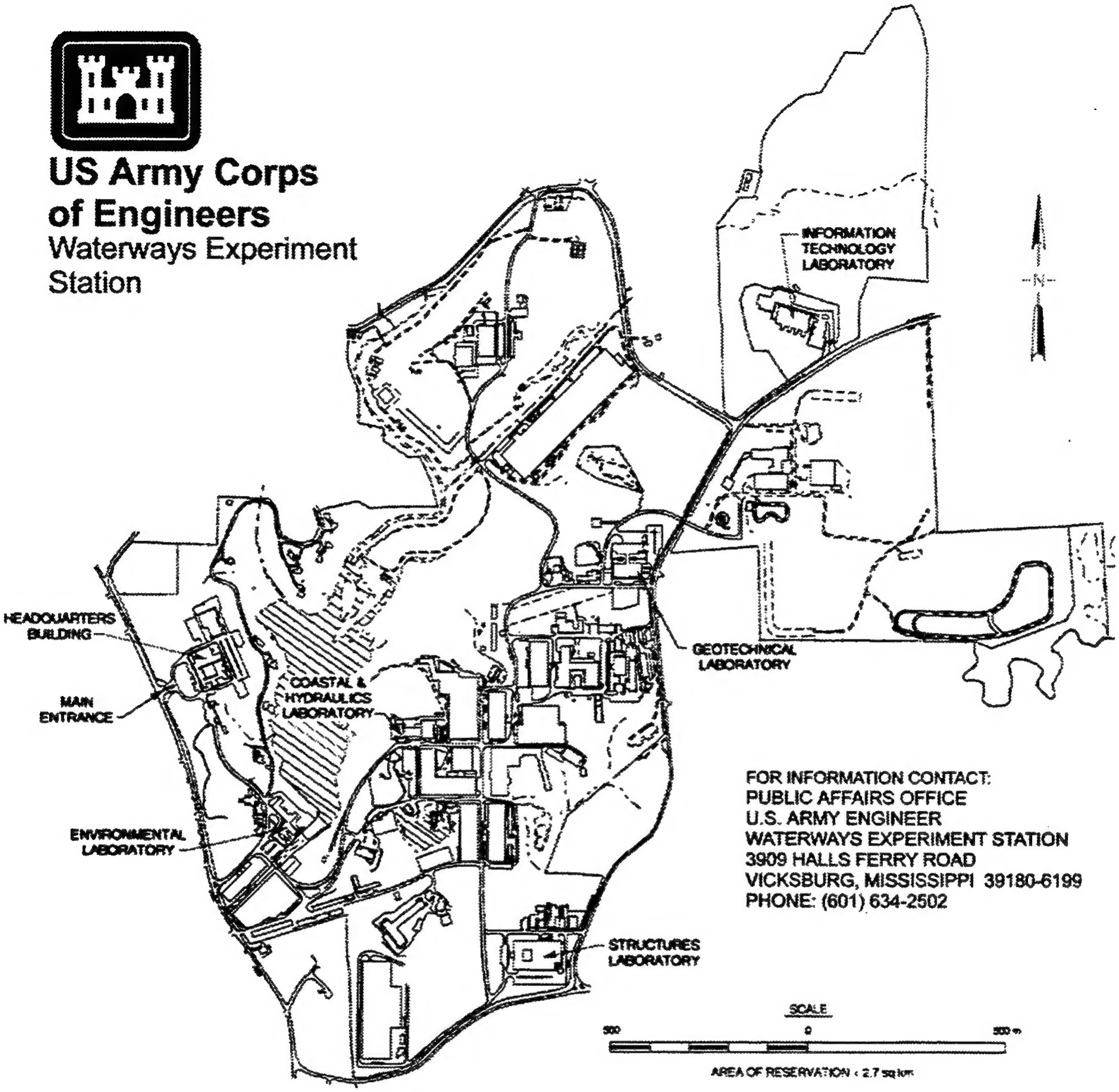

FOR INFORMATION CONTACT:
PUBLIC AFFAIRS OFFICE
U.S. ARMY ENGINEER
WATERWAYS EXPERIMENT STATION
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199
PHONE: (601) 634-2502

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Environmental Impact Research Program

US Army Corps of Engineers
Waterways Experiment Station

Riparian Vegetation Functions

Hydraulic Impacts of Riparian Vegetation; Summary of the Literature (TR EL-97-9)

ISSUE: Healthy riparian vegetation tends to stabilize streambanks, provides shade that prevents excessive water temperature fluctuations, performs a vital role in nutrient cycling and water quality, improves aesthetic and recreational benefits of a site, and is immensely productive as wildlife habitat. Concurrent with these benefits are impacts to the channel and floodway conveyance with subsequent sedimentation and stability impacts. Techniques that permit the quantification of these benefits and impacts are needed.

RESEARCH OBJECTIVES: The objectives of this research are to evaluate and document the accuracy and applicability of existing techniques for predicting hydraulic impacts in densely vegetated floodways, formulate new techniques as needed to accomplish these predictions, document the environmental benefits of riparian vegetation, and present guidance for selecting and maintaining riparian vegetation systems that optimize environmental benefits while minimizing hydraulic and stability impacts. This report addresses the first objective.

SUMMARY: Conventional flow formulas have proven satisfactory for predicting simple flow situations as they occur in compact channels with uniform boundary roughness. However, their ability to represent flow conditions when vegetation causes variable flow resistance, impacts upon turbulence, and momentum exchange between the channel and the floodplain is not documented. This report evaluates six existing techniques for predicting Manning's resistance coefficients for vegetated floodways. For the 19 reaches evaluated, the performance of the six methods tested was largely a function of the vegetation characteristics. In cases where the vegetation was limited to the banks or where densities were low, all of the methods performed reasonably well; the mean error of estimation for the six methods was +4.1 percent (24.7 percent absolute error). In cases with higher resistance, however, the six methods collectively underpredicted measured resistance by an average of 39.7 percent of the measured value. The shortcomings in these existing methods are identified and the groundwork for the development of new resistance relations laid.

AVAILABILITY OF REPORT: The report is available on Interlibrary Loan Service from the U.S. Army Engineer Waterways Experiment Station (WES) Library, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199; telephone (601) 634-2355.

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About the Author: Dr. J. Craig Fischenich is a research civil engineer at the WES Environmental Laboratory. Point of contact is Dr. Fischenich at (601) 634-3449.

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Preface

This report was prepared by personnel of the Environmental Engineering Division (EED), Environmental Laboratory (EL), U.S. Army Engineer Waterways Experiment Station (WES), as part of the Environmental Impact Research Program (EIRP). The EIRP is sponsored by Headquarters, U.S. Army Corps of Engineers (HQUSACE), and is assigned to WES under the purview of the EL. The HQUSACE Program Monitors were Ms. Cheryl Smith, Mr. Frederick B. Juhle, and Ms. Denise White. Dr. Russell F. Theriot, EL, WES, was the EIRP Program Manager.

Dr. J. Craig Fischenich, EED, was the author of this report. Messrs. Shawn Boelman, Thomas Cross, and Pulley Torres, all of EED, provided invaluable assistance in the compilation of information for this report. They also developed most of the figures and tables. Their support is gratefully acknowledged. Technical reviews by Mr. Elba A. Dardeau, Jr., and Dr. Mary Davis, both of the EL, are also gratefully acknowledged. The study was performed under the direct supervision of Mr. Norman R. Francingues, Chief, EED, and Dr. John Harrison, Director, EL.

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Conversion Factors Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
inches	25.4	millimeters

1 Introduction

Background

Hydraulic engineers have become increasingly involved in channel restoration projects and modifications to existing flood control projects as the conventional "flood control" ideology is replaced with a new "flood management" philosophy. The incorporation of vegetation into these projects is often mandated. Healthy riparian vegetation tends to stabilize streambanks, provides shade that prevents excessive water temperature fluctuations, performs a vital role in nutrient cycling and water quality, improves aesthetic and recreational benefits of a site, and is immensely productive as wildlife habitat.

Concurrent with these benefits are impacts to the channel and floodway conveyance with subsequent sedimentation and stability impacts. An estimate of the resistance to flow is required to conduct the hydraulic and geomorphic assessments necessary to quantify these impacts. Conventional flow formulas, including the Chezy, Darcy-Weisbach, and Manning equations, have proven satisfactory for predicting simple flow situations as they occur in compact channels with uniform boundary roughness. However, these conditions are seldom found in practice. Considerable variability of boundary roughness along the wetted perimeter caused by, among other things, vegetation on the banks and in the floodplain complicates evaluation of flow conditions due to variable flow resistance, impacts upon turbulence, and momentum exchange between the channel and the floodplain.

Although more complicated analyses are often warranted and sometimes performed, engineers typically evaluate river systems using one-dimensional, steady, gradually varied flow models. Depending upon the model used, engineers can account for the effects of dense vegetation by adjusting one or more of the following parameters: channel cross section, velocity coefficients, momentum coefficients, flow area subdivision, or resistance coefficients. Virtually no guidance exists for the adjustment of the first four parameters, and only limited guidance is available for the selection of resistance coefficients in vegetated floodways. Existing guidelines are entirely empirical and are limited to conditions under which the data were collected. These are seldom applicable to projects of concern as discussed in the literature review. Furthermore, adoption of guidance in the literature can be particularly misleading with regards to channel stability and can lead to design-deficient project failures.

The inability of engineers to calculate the effect of most types of vegetation upon channel conveyance prevents the development of projects that include appropriate vegetation within the riparian zone for environmental benefits, while providing the adequate conveyance. Thus, current practice calls for designs that use monotypic grasses that offer little habitat or aesthetic value and require strict maintenance requirements to keep the project within the narrow confines of known conveyance parameters. Guidance is needed to permit the design and maintenance of projects that optimize habitat value through proper species assemblages, planting arrangements, and maintenance regimes, while ensuring a stable, cost-effective design. Figure 1 shows an idealized cross section of a flood control channel with environmentally suitable vegetation in the floodway. No method in the literature would permit a design engineer to accurately assess flow depths and velocities in this channel.

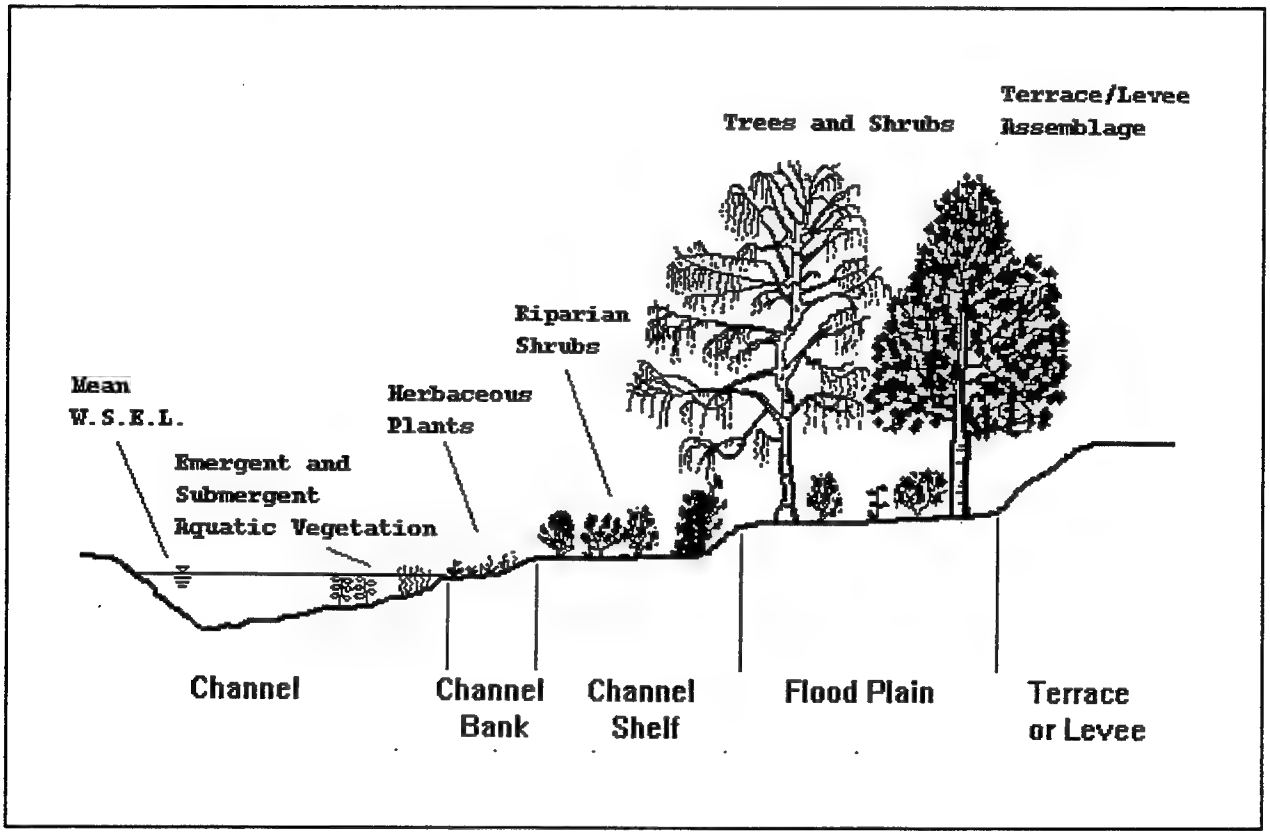

Figure 1. Flood control channel cross section

Research Objectives

The primary objective of this research is to develop a set of theoretically based hydraulic equations describing vertical velocity profiles in densely vegetated floodways. These equations will be formulated such that they exhibit the following characteristics: resistance will be related to readily defined, measurable characteristics of the channel, vegetation, and flow; resistance will be

described as a continuous function of the independent variables involved; and the function will be dimensionally homogeneous. Dimensionless coefficients describing channel, flow, and vegetation characteristics will be derived using existing data sets. Prediction functions for Manning's n will be formulated from the velocity profile equations under the assumption of steady uniform flow. Application of the derived equations to the solution of flow problems using a one-dimensional, steady, gradually varied flow model will be demonstrated and compared with existing methods.

It is intended that these equations will be generally applicable to the computation of water surface profiles, channel and floodplain velocities, and boundary shear for flows in alluvial channels. Specifically, they are intended to overcome difficulties associated with hydraulic computations and modeling for channels in which bars, islands, banks, or floodplains are heavily vegetated and the flow is fully turbulent.

Water surface profiles, channel and floodplain velocity distributions, and boundary shear stress distributions are computed for a variety of technical uses. Tens of thousands of profile analyses are performed each year for flood insurance studies, flood hazard mitigation investigations, and drainage crossing analyses (Hydrologic Engineering Center (HEC) 1986). Thousands more evaluations of velocity and shear distribution are conducted for channel stability analyses, sedimentation studies, and aquatic habitat investigations. In cases where appreciable vegetation exists on bars, islands, banklines, or floodplains, these analyses are handicapped by the lack of an adequate predictor of channel resistance. The need for the development of resistance relationships for vegetated floodways has been demonstrated by Arcement and Schneider (1989), Fischenich and Abt (1995), Kouwen (1988), and Yen (1992).

Approach

This research will address the objectives stated in the preceding section by accomplishing the following:

a. Development of a set of theoretically based hydraulic equations describing both horizontal and vertical velocity profiles in densely vegetated floodways.

b. Formulation of dimensionless coefficients describing channel, flow, and vegetation characteristics from existing data sets.

c. Relating the velocity profile prediction functions to Manning's n for increased applicability and acceptance by the engineering community.

d. Demonstration of the applicability of the derived equations to the solution of flow problems using a one-dimensional, steady, gradually varied flow model.

2 Literature Review

Background

"Modern" concepts of open-channel resistance did not surface until late in the 18th century, based largely upon work by Antoine Chezy (1718-1798). A flurry of activity ensued in the 19th century, from which emerged the three most well-known and commonly used resistance formula:

The Chezy Equation (1769):

$$V = C\sqrt{RS} \tag{1}$$

The Darcy-Weisbach Equation (1857):

$$V = \sqrt{\frac{8g}{f}}\sqrt{RS} \tag{2}$$

Manning's Monomial Equation (1889):

$$V = \frac{k_n}{n}R^{2/3}S^{1/2} \tag{3}$$

where

V = cross-sectional average velocity

C = dimensionless Chezy resistance coefficient

R = hydraulic radius

S = energy slope or momentum slope, depending upon approach followed

k_n = 1.486 for English units and k_n = 1 for SI units

f = dimensionless Weisbach resistance coefficient

n = Manning resistance coefficient

The collective works of Prandtl, Blasius, Hopf, von Karman, Nikuradse, Colebrook and White, Moody, Keulegan, and others have provided valuable insight into the relations between flow resistance and fluid mechanics. These studies have led to the development of relationships between resistance coefficients and logarithmic and power velocity distributions for flow in channels with rigid boundaries. The equations were summarized by Simons and Senturk (1976) as follows:

$$\frac{V}{U*} = c \log \left(a \frac{R}{k_s} \right) \tag{4}$$

for fully developed roughness where $(k_s U_*)/u > 70$;

$$\frac{V}{U*} = c \log \left(R_e \frac{\sqrt{f}}{b} \right) \tag{5}$$

for smooth flow where $(k_s U_*)/u < 5$; and,

$$\frac{V}{U*} = c \log \left(a \frac{R}{k_s} + R_e \frac{\sqrt{f}}{b} \right) \tag{6}$$

for flow in the transition. In these relations, V is the average cross-section velocity, U_* is the shear velocity, f is the Weisbach friction factor, R is the hydraulic radius, k_s is an equivalent sand roughness size, R_e is the Reynolds number, and a, b, and c are coefficients for which values have been proposed by several investigators. Because the mean velocity is the same for a channel regardless of the resistance relationship used, U_*, C, f, and n can be related from these equations as:

$$\frac{U*}{V} = \sqrt{\frac{f}{8}} = \frac{\sqrt{g}}{k_n} \frac{n}{R^{1/6}} = \frac{\sqrt{g}}{C} = \frac{\sqrt{gRS}}{V} \tag{7}$$

Contributing Parameters

For a fluid flowing through an alluvial channel, the hydraulic resistance to the flow is a function of many variables. Chow (1959) defined 10 significant factors contributing to flow resistance: (a) surface roughness, (b) vegetation, (c) channel irregularity, (d) channel alignment, (e) silting and scouring, (f) obstructions, (g) size and shape of the channel, (h) stage and discharge, (i) seasonal changes, and (j) suspended material and bed load. While this classification is easy to

comprehend, it prohibits direct analytical solution since many of the variables contributing to Chow's factors are dependent and the factors themselves are interdependent.

Rouse (1965) suggested that hydraulic resistance can be divided into four categories: (a) surface resistance, which is due to the viscous action on the channel boundary and is generally considered to be dependent on the Reynolds number of the flow and a relative roughness representing the physical characteristics of the boundary; (b) form resistance, which in alluvial channels is due to obstacles attached to the boundary that cause a form drag; (c) wave resistance, which is due to the free surface distortions, generating a net pressure difference between two cross sections; and (d) resistance due to flow unsteadiness. Yen (1992) further defined 20 specific variables affecting resistance under the four categories of fluid, flow, channel, and sediment.

Following the convention of identifying variables by grouping, Table 1 summarizes variables considered significant to resistance in vegetated alluvial channels and floodplains. The space-time variability of each of the parameters listed in Table 1 also influences flow resistance. Thus, for each parameter, P, the terms $\partial P/\partial x$, $\partial P/\partial y$, $\partial P/\partial z$, and $\partial P/\partial t$ must be considered.

Existing Techniques

In the United States, it is customary to express the flow resistance in terms of the resistance coefficient from Manning's Monomial Equation, *n*. Procedures for the computation or estimation of Manning's *n* in vegetated channels can be grouped into four general categories: direct measurement, analytical approaches, handbook methods, and effective area techniques. Direct measurement, though important for model and prototype (i.e., from high water marks) calibration and verification, is of little practical use for prediction and is not discussed.

Analytical approaches

Cowan (1956) proposed a procedure for estimating Manning's *n* that takes into account the contributions of various factors, including vegetation, to total flow resistance. The procedure, popularized by Chow (1959), Aldredge and Garrett (1973), and Arcement and Schneider (1989), is based upon the concept of linearity. As such, it assumes that the resistances induced by various contributing factors can be summed to establish total resistance. Cowan extended this theory to include resistance coefficients, namely Manning's *n*. Cowan's equation is as follows:

$$n = (n_b + n_1 + n_2 + n_3 + n_4)m \tag{8}$$

Table 1
Summary of Important Variables

Fluid Properties	
ρ	density
γ	specific weight
μ	dynamic viscosity
C_s	concentration of suspended sediment
Flow Variables	
g	gravitational acceleration
V	cross-sectional mean velocity
d	representative depth
S_w	water surface slope
U'	a nondimensional representation of velocity unsteadiness
U^*	a nondimensional representation of cross-sectional velocity variability
Channel Geometry Variables	
S_o	channel bed slope
A	channel area
P	wetted perimeter
W	water surface width
S	sinuosity
r	radius of curvature
λ	meander wavelength
Alluvial Boundary Variables	
ρ_s	sediment density
d_s	representative sediment diameter
G	sediment gradation
ξ	a nondimensional sediment shape factor
η	bedform type
δ_p	a nondimensional permeability coefficient
Vegetative Boundary Variables	
b	characteristic diameter or breadth of an individual vegetative element
h	average height of undeflected vegetation
h_d	average height of deflected vegetation
ϵ	a nondimensional representation of the vegetation "waviness" when submerged
J	flexural rigidity of the vegetation
l	measure of plant spacing for uniform distribution of plants over the bed
β	a dimensionless parameter characterizing the profile shape of an individual plant
ω	frontal density of vegetation field

where

n = Manning's n value

n_b = base n value

n_1 = addition for surface irregularities

n_2 = addition for variation in channel cross section

n_3 = addition for obstructions

n_4 = addition for vegetation

m = ratio for meandering

Using Cowan's approach, an engineer selects a base value for n, then increases this value by adding adjustments for each of the factors described above. A coefficient for meandering is applied to the additive factors, and the resultant Manning's n value is used to calculate hydraulic parameters for the channel using whatever procedure the engineer chooses. Roughness values for channels and floodplains are determined separately since the composition, physical shape, and vegetation of a floodplain can be quite different from those of a channel. Compositing techniques are used to develop a composite roughness value for the channel/floodplain combination, or the conveyance for each can be calculated and summed. Tables 2 and 3 provide the recommended adjustment factors for vegetation. The selection of an appropriate adjustment value from the tables is very subjective. Verification of the table values could not be located in any literature.

Petryk and Bosmajian (1975) developed a method of analysis of the vegetation density to determine the roughness coefficient for a densely vegetated floodplain. The method is based upon Cowan's, but explicitly includes vegetation density in the computations. By summing the forces in the longitudinal direction of a reach and substituting in the Manning formula, they developed the following equation:

$$n = n_0 \sqrt{1 + \left(\frac{C_* \Sigma A_i}{2gAL}\right)\left(\frac{1.49}{n_0}\right)^2 R^{4/3}} \qquad (9)$$

where

n_0 = Manning's coefficient, excluding effect of vegetation

C_* = effective-drag coefficient for vegetation in direction of flow

$\sum A_i$ = frontal area of vegetation blocking flow in reach, sq ft

g = gravitational constant, ft/s^2

A = cross-sectional area of flow, sq ft

L = length of channel reach being considered, ft

R = hydraulic radius, ft

Table 2
Adjustment Values for Factors That Affect the Roughness of a Channel (Modified from Aldredge and Garrett 1973)

Amount of Vegetation (n_4)	*n* Value Adjustment	Example
Small	0.002-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrowweed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation
Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemmy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees in the dormant season, growing along the banks, and no significant vegetation is evident along the channel bottoms where the hydraulic radius exceeds 2 ft
Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8- to 10-year-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft; bushy willows about 1 year old intergrown with some weeds along side slopes (all vegetation in full foliage), and no significant vegetation exists along channel bottoms where the hydraulic radius is greater than 2 ft
Very Large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; bushy willow trees about 1 year old intergrown with weeds along side slopes (all vegetation in full foliage), or dense cattails growing along channel bottom; trees intergrown with weeds and brush (all vegetation in full foliage)

Equation 9 presents the n value in terms of the boundary roughness n_0, the hydraulic radius R, an effective-drag coefficient C_*, and the vegetation density $\Sigma A_i/AL$. According to Arcement and Schneider (1989), effective-drag coefficients for densely wooded floodplains can be approximated from the relation $C_* = 22 - (3.75)\,R$. Caution should be exercised in applying this relation, however. It yields drag coefficients considerably higher than those suggested by most researchers (values ranging from 0.5 to 1.5 are commonly cited). This discrepancy is probably explained by differences in the flow conditions; Arcement and Schneider's relation was likely developed for flows with very low Reynolds number, whereas the others were formulated for fully turbulent flow. The total boundary roughness n_o is determined from:

$$n_0 = n_b + n_1 + n_2 + n_3 + n_4 \qquad (10)$$

Table 3
Adjustment Values for Factors That Affect the Roughness of Floodplains (Modified from Aldredge and Garrett 1973, Table 2)

Amount of Vegetation (n_4)	n Value Adjustment	Example
Small	0.001-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrowweed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation
Medium	0.011-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemmy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees in the dormant season
Large	0.025-0.050	Turf grass growing where the average depth equals the height of the vegetation; 8- to 10-year-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft; mature row crops such as small vegetables, or mature field crops where depth of flow is at least twice the height of the vegetation
Very Large	0.050-0.100	Turf grass growing where the average flow depth is less than half the vegetation height; moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is below branches; mature field crops where depth of flow is less than the height of the vegetation
Extreme	0.100-0.200	Dense bushy willow, mesquite, and saltcedar (all vegetation in full foliage): heavy stand of timber, few down trees, depth of flow reaching branches

The definition of the roughness factors n_b and n_1 through n_3 are the same as those for Cowan's method. The n_4' factor is for vegetation such as shrubs, brush, and grass on the surface of the floodplain that could not be measured directly in the vegetation density term of Equation 9. The n_4' factor should be restricted to the small-to-medium range in Table 2 because a tree canopy prohibits dense undergrowth.

Cowan's procedure as modified by Petryk and Bosmajian is applicable to floodplains with stands of mature trees, with little undergrowth. Although Petryk and Bosmajian indicate that vines and other undergrowth be considered in the $\sum A_i$ term, they provide no C_* values for this vegetation. The mechanics of flow through shrubby vegetation are considerably different from those for flow around tree trunks. For flow over and through herbaceous or shrubby vegetation, other techniques should be used.

A number of authors associated with the U.S. Department of Agriculture, Agricultural Research Service including Cox and Palmer (1948) and Ree and Palmer (1949) summarized research of flow in vegetated channels conducted by Soil Conservation Service (SCS) researchers from 1935 to 1943 at Spartansburg, GA, and Stillwater, OK. The investigators found that most flow data for a particular grass, when plotted with *n* as a function of the product of velocity and hydraulic radius, would fall approximately along a single line. The most frequently reproduced graph from these experiments (Figure 2) summarizes the *n* - VR curves for five "classes" of vegetation, each class considered to have similar properties. Use of this graph is referred to as the SCS method.

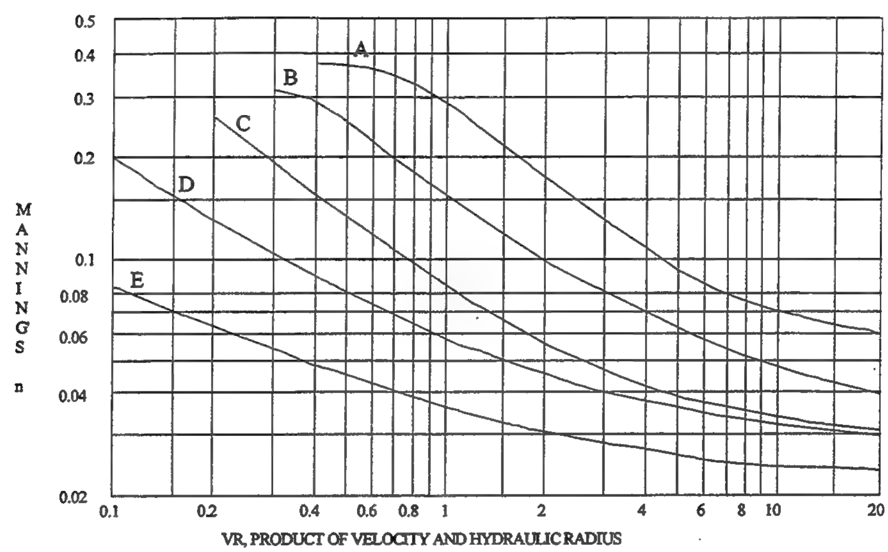

Figure 2. *n* - VR relationships for grass cover

Seven different herbaceous species were tested in these early investigations. They include alfalfa (*Medicago sativa*), long and short Bermuda grass (*Cynodon dactylon*), blue grama (*Bouteloua gracilis*), buffalo grass (*Buchloe dactyloides*), weeping lovegrass (*Eragrostis curvula*), and a native grass mixture consisting of little bluestem (*Andropogon scoparius*), Indian grass (*Sorghastrum nutans*), switch grass (*Panicum virgatum*), blue grama, and side-oats grama (*Bouteloua curtipendula*). Ree and Crow (1977) investigated wheat, sorghum, cotton, and lespedeza for smaller slopes and have published additional *n* - VR curves for these cases.

While they are not derived from basic fluid mechanics principles, the *n* - VR curves can be accurately reconstructed using the relative roughness method. Most investigators have claimed that the *n* versus VR relationship is practically independent of channel slope and shape. This claim is probably true in the turbulent flow regime, but not in the laminar or transition flow regimes. In discussing the *n* - VR method, Ree and Crow (1977) point out that the original investigators responsible for the development of the *n* - VR approach were concerned that their data had been applied outside the intended range. The original tests were conducted on channels with slopes greater than 3 percent, and numerous studies have shown that the *n* - VR curves are not applicable for smaller slopes (Eastgate 1966; Gwinn and Ree 1980; Kouwen 1988; Ree and Crow 1977). However, encouragement to use the method outside its intended range is pro-vided by the "Handbook of Channel Design for Soil and Water Conservation" (U.S. Soil Conservation Service 1954). This handbook provides design curves for slopes as low as 0.01 percent, although the data do not support them.

Gwinn and Ree (1980) duplicated some of the Stillwater experiments in a channel previously used to test grasses. After 25 years without maintenance, the channel characteristics had changed with the encroachment of brush and trees. For the smaller VR values, Manning's *n* had decreased because grass no longer grew where brush was present. However, for larger VR values, Manning's *n* had increased. The effect of the brush and tree encroachment was to reduce the flow capacity by 29 percent and to reduce the permissible velocity in the 6-percent-slope reach to 1.1 m/s (3.6 ft/s).

Chen (1976) conducted tests on two species of turf that can be sodded; Kentucky bluegrass and Bermuda grass. Average turf height was maintained at approximately 3 in. (76 mm) for the tests, which were conducted in a laboratory flume. Seven bed slopes for each turf were tested. The slopes tested were 0.1 percent (0.001), 0.5 percent (0.005), 2 deg (0.035), 5 deg (0.087), 6:1 (0.164), 3:1 (0.316), and 1.5:1 (0.555). Rather than adopt the *n* - VR relationship proposed by the SCS investigators, Chen postulated that resistance was a function of the Reynolds number R_e. He found that in the laminar flow range, a relationship between the Darcy-Weisbach friction factor and Reynolds number existed for each bed slope tested. The f value increased with the bed slope, but decreased with the Reynolds number (or unit discharge for constant water temperature). By ignoring negligible factors such as raindrop impact and the slope limits, Chen determined that the friction coefficient for shallow flows over natural turf surfaces can be expressed as:

$$f = \frac{510{,}000(S_0^{0.0662})}{R_e} \tag{11}$$

As long as the flow under study is in the laminar flow range (i.e., approximate $R_e = 10{,}000$ for $S_o = 0.01$ and $R_e = 1{,}000$ for $S_o = 0.555$), Equation 11 can be used to evaluate the friction coefficient for shallow flows over Kentucky bluegrass and Bermuda grass surfaces. However, whether or not Equation 11 is also applicable to other species of turf other than Kentucky bluegrass and Bermuda grass was not experimentally investigated.

In addition to slope and channel shape concerns, the applicability of the n - VR method is hindered by the difficulty of choosing the appropriate curve. Since the curves were developed for specific species of vegetation, they are applicable only to that specific species, in the same condition, and for the identical channel slope. Extrapolating the relationships to other species or even the same species in a different condition can result in gross overprediction or underprediction of Manning's n. Much of the uncertainty in selecting a curve could be eliminated by defining the biomechanical properties of vegetation rather than the species. Thus, a curve should be selected based upon the vegetation's height, density, and stem rigidity rather than observed similarity of physical character.

Kouwen and Unny (1973) proposed an improvement to the SCS method by suggesting that vegetation be classified on the basis of its flexural rigidity, defined as the relative stem density M times the modulus of elasticity of the vegetation E times the second moment of the cross-sectional area of the stems I. For natural vegetation encountered in field conditions, it is not possible to determine the values for M, E, and I because of the great variability that exists for the vegetation. Instead, the combined effect of the product MEI is used as a single parameter that reflects the overall resistance to deformation of the lining as a result of a flow passing over it. Kouwen, Li, and Simons (1981) present tables of MEI values for several species of vegetation and guidance for selecting the appropriate SCS curve based upon MEI value. Table 4 summarizes this information.

Table 4
Classification of Vegetal Covers (After Kouwen, Li, and Simons 1981)

Retardance Class	Cover Type	Condition	MEI, Nm^2
A	Weeping lovegrass	Excellent, 760 mm tall	200
	Rhodes grass	Excellent, 690 mm tall	140
A/B	Kikuyu grass	Excellent, 420 mm tall	47
	Bermudagrass	Good, 300 mm tall	17
	Long prairie grass mix	Good, unmowed	20
	Weeping lovegrass	Good, 600 mm tall	30
B	Lespedeza sericea	Good, 600 mm tall	10
	Alfalfa	Good, uncut, 275 mm tall	4
	Weeping lovegrass	Good, mowed, 330 mm tall	6
	Blue grama	Good, uncut, 330 mm tall	8
	Bluegrass	Unknown, 340 mm tall	18
	Dallas	Uncut, 760 mm tall	20
B/C	African star	Unknown, 290 mm tall	4.6
	Bermuda grass	Good, mowed, 150 mm tall	2.0
	Common lespedeza	Good, uncut, 280 mm tall	3.0
D	Centipede grass	Very dense, 150 mm tall	2.0
	Bermuda grass	Good, mowed, 64 mm tall	0.15
	Common lespedeza	Excellent, uncut, 114 mm tall	0.10
	Buffalo grass	Good, uncut, 75-150 mm tall	0.16
	Grass-legume mixture	Good, uncut, 100-125 mm tall	0.7
	Lespedeza sericea	Very good, cut, 50 mm tall	0.005
	Kikuyu	Unknown, cut, 107 mm tall	0.17
	Kentucky bluegrass	Unknown, mowed, 75 mm tall	0.10
E	Bermuda grass	Good, mowed, 38 mm tall	0.03

Kouwen (1988) presented two methods for estimating the value of *MEI*. The first method computes stiffness based on grass length. The values of *MEI* calibrated by comparing the flow resistance from natural grass linings are much larger for long grass than for the same grass after cutting, which is due to the way the deflection of the vegetative mat under shear is defined, namely, the vertical compression of the material as given by the ratio k/h. Kouwen found the correlation between *MEI* and the grass length to be very high (95 percent) for green grasses. For dormant grasses, the correlation was weaker (83 percent). Equations 12 and 13 give the relationship between *MEI* and h for green and dormant vegetation, respectively.

Green vegetation:

$$MEI = 319h^{3.3} \tag{12}$$

Dormant vegetation:

$$MEI = 25.4h^{2.26} \tag{13}$$

Equations 12 and 13 show that when long grass is cut, the stiffness (*MEI*) of the remaining stubble is less than the stiffness of the grass before it is cut. Considering that the most flexible material is removed and the most rigid (stubble) remains, this result seems opposite to what should be the case. However, the phenomenon is explained by the fact that as the vegetation bends, it becomes a denser medium, and gradually exhibits greater resistance to bending. The stubble is embedded in the bottom part of the grass mat, and the top layer adds to the overall apparent stiffness. While to the casual observer, the longer vegetation appears not as stiff as its mowed bottom layer, its hydraulic effect is to display greater stiffness due to its increased density as it is compressed.

The board drop test first suggested by Eastgate (1966) and adopted by Kouwen (1988) is the second method for determining *MEI*. This method provides an objective approach to quantifying the biomechanical properties of herbaceous vegetation channel linings. Eastgate reported on a number of tests carried out over a natural grass lining installed in a tilting flume. He suggested the use of the board drop test as a means to determine which of the n - VR curves applies to a given vegetative lining. The test consisted of standing a 1,829- by 305-mm board weighing 4.85 kg vertically on one end and allowing it to fall freely. The board rotates about the end in contact with the ground. When the board hits the grass, it slides lengthwise in the direction of rotation, imparting a friction force which, along with the weight of the board, deflects the grass in a manner similar to flowing water. Kouwen repeated this process on grassed areas in Ontario. Eastgate and Kouwen recorded the distance between the ground and the bottom edge of the fallen end of the board each time, and were thus able to develop relationships between vegetation type and deflection.

Kadlec (1990) hypothesized out that for emergent wetland vegetation, resistance should be computed on the basis of the sum of the drag on single objects (stems) since they are typically spaced several diameters apart. In wetlands, flows are typically in either the laminar or transition region rather than turbulent, so Kadlec abandoned the use of Manning's equation as a means to characterize flow resistance. Furthermore, he stated that vegetation drag controls resistance in wetlands. He suggested that the appropriate relation to use to describe flow in wetlands is the drag expression for isolated submerged objects:

$$S = C_D \, a \frac{V^2}{2g} = X \, \frac{V^2}{2g} \qquad (14)$$

where

S = friction slope

C_D = drag coefficient

a = frontal area of vegetation per unit volume

V = average velocity

X = a lumped resistance coefficient $C_D a$

Kadlec indicated that in lieu of drag coefficients, C_D, for each species of vegetation, drag coefficients for single cylinders can be combined with measured values of vegetation area to yield a reasonable prediction of flow rate. While Kadlec achieved a good correlation with measured data using this technique, the wetland vegetation used, namely *Spartina* and *Carex*, is probably more similar to cylindrical objects than are most species of vegetation.

Handbook methods

Establishment of flow resistance with procedures that do not rely on direct measurement or numerical analysis are referred to herein as the "handbook method." Included in this category are the familiar tables of roughness values and the estimation of roughness values based upon visual comparison. The handbook methods are the most widely used approaches for the evaluation of channel roughness. Chow (1959) is regarded as the pioneer of this approach to solving for flow resistance. The tables of Manning's n values published in his book are likely the most common source of information for the selection of a channel and floodplain roughness values. Chow provides minimum, normal, and maximum values of Manning's n for conduits, lined canals, and natural channels. Also in this reference are photographs of 24 channels with captions indicating a Manning's n value and a qualitative description of the channel. Chow's work still represents one of the most comprehensive attempts at describing roughness for a wide range of channel conditions. Of the 111 channel and floodplain types listed

in Chow's table, only 27 include vegetation, and only 11 of the 24 photographs show evidence of vegetation. Many other tables of roughness values have been developed. However, most of these rely upon data from laboratory analyses or from observations made on artificial channels with little or no vegetation and, thus, are of only limited value for the purpose of this study. Others are based largely upon the same data Chow used in his analysis, and thus offer little additional insight.

The visual comparison approach relies on the use of "calibrated photographs" and associated roughness of certain channels that can be found in several references as a means of estimating roughness values for the channel of interest. The work by Chow described above is an example. Aldridge and Garrett (1973) present photographs of selected Arizona channels and floodplains having known roughness coefficients. Included with the photographs are descriptions of channel geometry and the roughness factors involved in assigning a Manning's *n* value.

Among the publications presenting a pictorial accounting of channels with published roughness values, Barnes (1967) is perhaps the most recognized. Barnes presents color photographs and descriptive data for 50 stream channels, nearly all of which have vegetation on the banks. Unfortunately, nearly all of the data presented in Barnes' report pertains to the main channel only. Where overbank flow existed, it was omitted from the calculations. An exception was made in the case of the Rolling Fork River in Kentucky. Overbank *n* values were calculated for flows 5 to 7 ft[1] deep in a floodplain having a "fairly dense stand of trees as much as 6 in. in diameter." The identified *n* value for the overbank was 0.097, whereas for the main channel it was 0.046.

The information presented in Barnes' document is much more detailed than that provided by Chow and other earlier works. In addition to a qualitative description of the channel, Barnes provides quantitative information, including cross sections and hydraulic elements. Because of the advantages provided by improved photography and better channel descriptions, Barnes' work is preferred over earlier efforts. For 9 of the 50 streams, Barnes analyzed more than one discharge, which provides some insight as to the variability of Manning's *n*.

Hicks and Mason (1991) is the most comprehensive reference for the visual comparison method. The format of this book is very similar in nature to that of Barnes (1967) in that color photographs and descriptions of channels are provided along with roughness values calculated from field measurements. Improvements over Barnes are derived from the use of multiple photographs for each reach, the presentation of bed material gradations, a summary table that includes discharge, water surface slope, friction slope, area, expansion expressed in percent, hydraulic radius, mean velocity, computed values for both Manning's *n* and Chezy's *C*, and an estimate of error for the computations. Most significantly, multiple discharges were evaluated for each reach.

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[1] A table of factors for converting non-SI units of measurement to SI units is presented on page viii.

Hicks and Mason presented information for 78 river reaches in New Zealand encompassing a broad range of conditions that are quite representative of conditions found elsewhere in the world. Like Barnes, Hicks and Mason avoided computation of flow resistance in floodplains, although their work does provide some insight as to the contribution of bank vegetation to channel roughness. For each reach, plots of the variation of Manning's n and Chezy C with discharge are presented. Although in many cases it is difficult to attribute the roughness changes to vegetation, the reader is provided some sense of the magnitude of influence due to bank vegetation.

Arcement and Schneider (1989) presented photographs for 15 densely vegetated floodplains for which roughness coefficients have been verified. This work represents the only visual comparison method known to this author in which an attempt was made to identify the specific roughness contribution due to vegetation. Using the general procedure suggested by Cowan and the vegetation-density method proposed by Petryk and Bosmajian, Arcement and Schneider used measured vegetation density in the floodplain and an effective drag coefficient for vegetation to calculate the contribution of vegetation to the total roughness value. Values for Manning's n for the sites ranged from 0.10 to 0.20. The contribution due to vegetation ranged from 0.065 to 0.145, accounting for 64 to 81 percent of the total n value.

Thomas (1994)[1] stated that "the handbook methods are probably more dependable as sources of n values than the analytical methods are because the compositing is included in the field observation." While the difficulties in accurate compositing are significant, it should be pointed out that there are many limitations to the handbook methods as well. It is very difficult in many cases to adequately observe and describe the condition of the channel. Even Barnes (1967) pointed out that the use of his book is primarily for the development of experience through familiarization with the appearance, geometry, and roughness characteristics of certain channels. While the handbook methods are convenient and offer the advantage of implicitly compositing channel resistance, they offer little utility in cases where vegetation is present or where floodplain flows are anticipated. Only a couple dozen such circumstances are addressed in the collective works of the handbooks cited, so identification of an appropriate comparison channel is frequently difficult or impossible.

Effective area techniques

Some practicing engineers, lacking guidance for the evaluation of densely vegetated floodways, simply eliminate the vegetated portion from the cross section for the purpose of hydraulic investigations. Recent research by Fukuoka and Fujita (1993), as well as other Japanese investigators, indicates that this procedure may be an effective means of computing water surface profiles for large rivers with densely vegetated floodplains.

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[1] Thomas, W. A. (1994). "Methods for predicting n-values for the Manning equation," Unpublished draft technical report, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Fukuoka and Fujita (1993) conducted laboratory investigations to quantify the momentum losses (and associated resistance) that occur as a result of fluid mixing at the interface between high-velocity main channel flows and low-velocity flows associated with an inundated floodplain with dense vegetation. The researchers developed boundary mixing coefficients from laboratory data and used these values to compute the shear stress acting on the interface between the main-channel and vegetated floodplain flows. Applying compositing techniques that exclude the vegetated portion of the channel but which apply an effective shear force at the interface, Fukuoka and Fujita were able to compute Manning's *n* values that yielded fair approximations of average velocities and water surface levels in a reach of the Ishikari River for two floods.

Several problems are inherent with the effective area approach. Among these, the most significant are that (a) resistance coefficients must be assigned to the vegetation/water perimeter, and no guidance exists, (b) momentum transfer occurs between the vegetated and unvegetated portion of the channel, and means to account for this are yet to be formulated, (c) hydrodynamic assessments are not possible since the vegetated areas eliminated from the cross section often provide substantial storage and/or flood wave attenuation, and (d) guidance for the discretization of the sections has not been postulated.

Despite these shortcomings, effective area techniques arguably have a basis in boundary layer theory. Fluid mechanists frequently approximate solutions to problems dealing with flow over rough surfaces by computing a displacement or momentum thickness which, when eliminated from the computational region, allows the outer flow region to be treated as a frictionless flow with the same mass flux as the actual flow. Atmospheric scientists have adopted a similar reasoning when evaluating flow through crops and forests for various types of analyses, defining a displacement height to a plane of zero velocity, an effective roughness length above this height, and vegetation and bulk drag coefficients to further define the shear induced by the vegetation. They use these coefficients in formulating turbulent velocity profile equations based upon the work by Prandtl (1904).

Atmospheric science approaches

Near-ground velocity profiles, boundary layer thickness, and shear velocity profiles for wind flow over and through vegetation are of interest to meteorological scientists and fluid mechanists involved with wind power generation, soil erosion control, and crop management, among other things. As a consequence, several studies have been made of the behavior of winds inside and directly above forest canopies and crops (Bayton et al. 1965; Cooper 1965; Denmead 1976; Dolman 1986; Grant 1984; Tourin and Shen 1966; etc.). Much of this data is accumulated in periodic issues and books (Geiger 1950; Monteith 1976; Raupauch and Thom 1981), and Meroney (1993) summarized the literature related to this topic.

Above a vegetation canopy, but within the surface layer (the lowest 10 percent of the atmospheric boundary layer, or approximately the lowest 100 m of the atmosphere), the mean wind-speed profile is commonly described by logarithmic expressions (Counihan 1975). For situations when stratification has only a minor influence, a modified logarithmic law has been proposed (Meroney 1993):

$$U(z) = \left(\frac{U_*}{k}\right) \ln_e\left[\frac{(z - d + z_o)}{z_o}\right] \qquad (15)$$

where

$U_* = (T/\rho)^{1/2}$ is the surface friction velocity

d = zero-plane displacement

k = Von Karman's shear layer constant

z_o = surface roughness

Figure 3 graphically presents the parameters in Equation 15. The displacement thickness, d, is important for tall roughness elements such as agricultural crops, forests, and cities. When the roughness elements are short, such that $z_o <$ 0.2 m, one can set $d = 0$. The parameters can be determined from representative

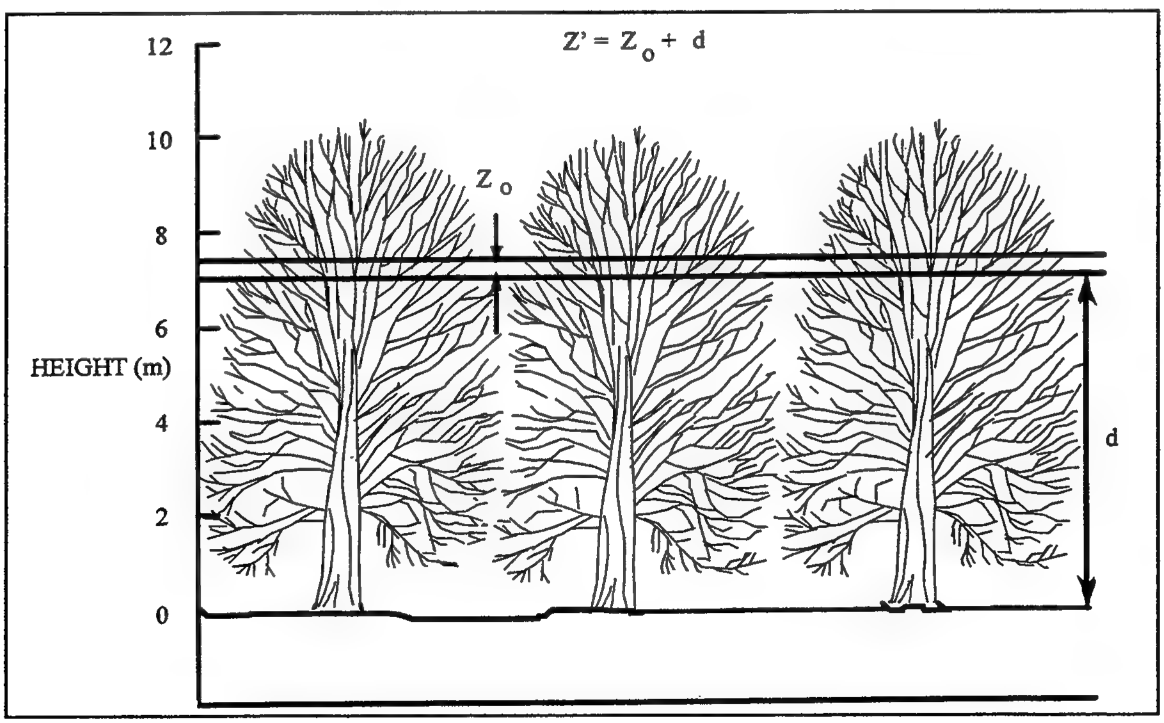

Figure 3. Schematic of d and z_o parameters

field measurements or models. Fitting an expression that permits three free parameters to field measurements of wind speed in agricultural canopies is not trivial, and it is not uncommon for some least-square fitting routines to produce negative displacement heights (Meroney 1993).

To avoid negative displacement height values, it is customary to assume the von Karman constant $k = 0.4$, to prespecify displacement height as some fraction of the forest canopy depth (say $d = 0.67\ h$) and to solve for friction velocity and surface roughness height by fitting Equation 15 to measured data. Surface roughness estimates have been estimated by many scientists for flow data obtained over different crops and forests. There is a wide variance in results even for flow over the same surface. Experimentalists frequently fail to obtain data above the wake region of individual roughness elements ($z > 1.5\ h$); sometimes the data are taken during nonneutral conditions; and often upwind nonhomogenuities distort the measured profiles. Summaries of tabulated data for displacement thickness are presented in Table 5.

Table 5
Summary of Roughness Length Data (modified from Meroney 1993)

Surface Type		Sutton (1949) Geiger (1950)	Priestly (1959)	Davenport (1965)	Counihan (1975)	Simiu and Scanlan (1978) Snyder (1981)
Sand		0.03-0.1	0.03			0.01-0.1
Mown Grass -	1.0 cm	0.1-0.2	0.2			0.1-1.0
	3.0 cm		0.7			
	4.5 cm	0.7-2.0	1.7-2.4			
Flat Open Country		2.0-3.0		1.75-6.5		
Low Grass, Steppe		1.0-4.0			0.1-20.0	1.0-4.0
Fallow Field		5.0			0.1-20.0	2.0-3.0
High Grass		3.0-9.0	3.7-9.0		0.1-20.0	4.0-10.0
Paletto		3.0-14.0				10-30
Pine Forest (h = 15 m)		20			100-150	90-100
Towns, Suburbs				20-90	100-150	20-40

An alternative empirical approach to describe the wind variation with height is a simple power law of elevation. It is widely used in describing the wind shear in the atmospheric surface and internal boundary layers in view of its simple format and engineering expediency. The general form of the expression used is (Meroney 1993):

$$U(z)/U_{ref} = (z/z_{ref})^{\alpha} \qquad (16)$$

where

U_{ref} = reference wind at a reference height z_{ref}

α = power law index (exponent)

The effect of turbulence induced by the surface roughness upon the wind shear is accounted for by the magnitude of the power law index, which is normally smaller than unity but larger than zero. Often the power law index is determined empirically by fitting Equation 14 to measured data; however, it is also possible to match the magnitude of predicted velocity and shear at a specified height and relate the index, α, to logarithmic parameters (z_o, d, and L_{mo}). For neutral flow the expression is simply (Meroney 1993):

$$\alpha = \frac{Z_m}{(Z_m d + z_o) \ln[(Z_m - d + z_o)/z_o^2]} \quad (17)$$

where Z_m is the matching or midheight over which both profiles are presumed valid.

Empirical expressions that relate power law index and surface roughness length have been proposed by Counihan (1975) and Baron (1982). Counihan's expression was developed by fitting logarithmic and modified logarithmic profiles to 70 sites over data to a height of 100 m:

$$\alpha = 0.096 \log_{10}[z_o] + 0.016(\log_{10}[z_o])^2 + 0.24 \quad (18)$$

for $0.001 \leq z_o \leq 5$. Baron fit a similar relationship to the nomogram proposed by Davenport (1965) such that:

$$\alpha = 0.125 \log_{10}[z_o] + 0.0004/z_o + 0.336 \quad (19)$$

for a roughness range $0.01 \leq z_o(\mathrm{m}) \leq 5.5$. However, the two functions produce significantly different estimates. For example, Baron's expression produces power index values 17 to 38 percent greater than Counihan's expression over the range from smooth to rough. This variation may simply be the result of using different data sets, the influence of stratification, or it may be that displacement height was not considered in a similar manner for the two data sets. Baron (1982) examined a wide cross section of field and laboratory data and created figures relating the power law index to element and roughness height.

For flow within the vegetation canopy, different profiles have been proposed by meteorologists using first order closure models that specify a simple eddy diffusivity, K, and a drag coefficient, C_d, for constant foliage distribution:

$$u/u_h = [(\sinh \beta\xi)/\sinh \beta]^{0.5} \quad \text{(Cowan 1968) (20)}$$

$u/u_h = \exp[-\beta(1 - \xi)/2]$ (Inoue 1963; Cionco 1965) (21)

$u/u_h = [(\cosh \beta\xi)/\cosh \beta]^{0.5}$ (Massman 1987) (22)

where

$\xi = z/h$

u_h = mean horizontal wind speed at top of canopy, h

β = a maximum value of foliage area density and extinction coefficient:

$$\beta = [2C_d LAI/(\sigma\mu)]^{0.5} \quad (23)$$

which is a combination of the drag coefficient, C_d, the leaf-area-index, LAI, a measure of foliage distribution, σ, and a normalized eddy diffusivity, $\mu = K/hu = K_h/hu_h$. Only the expression proposed by Massman is consistent with the observed zero wind gradient within the lower region of the canopy. Other authors have produced velocity profiles for nonconstant foliage distributions and using higher order turbulence closure (Albini 1981).

Once a velocity distribution model is specified, it is possible to solve by iteration for shear stand drag coefficient, $C_f = 2(U_*/U_h)^2$, displacement height, d, and surface roughness, z_o, parameters useful to characterize above canopy flow dynamics as functions of $C_d LAI$ and foliage structure. Massman (1987) concluded that $C_d LAI$ values from 0.25 to 0.50 characterize most full foliage canopies. Over this range, almost any within-canopy model gives results very close to the following expressions:

$$0.10 < z_o/h < 0.13 \quad (24)$$

$$0.67 < d/h < 0.75 \quad (25)$$

$$0.17 < C_f < 0.20 \quad (26)$$

Discussion

At the present stage of knowledge, selection of an n value requires an estimate of the resistance to flow in a given channel, which is a matter of intangibles. To veteran engineers, this means the exercise of sound engineering judgment and experience; for beginners, it can be no more than a guess, and different individuals will obtain different results. The presence of vegetation in the channel, on the banks, or in the floodplain can significantly complicate the prediction of channel stability and hydraulic characteristics. Fischenich and Abt (1995) and Yen (1992) are among authors who have recently summarized the state of the art in the prediction of resistance values in alluvial channels. Both

concluded that additional work is required, particularly in the case of channels with appreciable vegetation.

Flow through and over woody vegetation is complicated by the fact that several thresholds exist at which the resistance components change because of the response of the plant(s). At low flows, resistance is primarily the result of form losses from drag induced by the trunks/stems of the vegetation. As flow increases and reaches the height of the canopy, the resistance increases because of increased drag generated by the plant's stems and leaves. Resistance begins to decrease only when the plant yields by deforming to present a smaller area to the flow. If the force of the flow continues to increase, additional decreases in resistance may result from failures of the plant's leaves and stems and, at some point, the entire plant may be uprooted. When fully submerged, resistance consists not only of the form loss due to drag, but also of the viscous shear stress on the boundary of the vegetation field.

Procedures for the computation or estimation of flow resistance can be grouped into five categories: those based upon direct measurement, those based upon analytical solution, those following one of the popular handbook methods, effective area techniques, and meteorological approaches. Direct measurement may be the most accurate means of obtaining the estimate. In practice, however, measurement of the hydraulic parameters of a channel for the full range of flows for which resistance values are sought is seldom possible. Furthermore, the considerable variability of resistance for even a single location and discharge value make predictions of resistance based upon direct measurement somewhat suspect.

The analytical approaches are equally handicapped. Cowan's procedure offers an easily understood accounting method, but relies on interpretation of qualitative descriptions for the assignment of resistance values. It is also predicated on the verity of the linear superposition concept. The SCS method has been widely used to estimate resistance in grass-lined channels. The procedure is easy to comprehend and apply. Unfortunately, the applicability of the method is limited by the range of slopes over which data were collected and by the types of vegetation used. The tests were developed to evaluate the stability and hydraulics of grassed waterways in agricultural settings. Its use for flood control channel and floodplain analysis is limited to those conditions where the slope exceeds 3 percent and the vegetal cover is monotypic and similar to one that was tested. Such conditions are seldom encountered.

The vegetation density method of Petryk and Bosmajian and the *MEI* analyses proposed by Kouwen are the principal analytical procedures that explicitly include the geometric and physical properties of the vegetation in the analysis of flow resistance. These methods, though not yet substantially verified, appear to provide a sound process for the assessment of resistance due to vegetation. In light of the abundant number of vegetation species that might be employed in a channel or floodplain project, methods such as these of generalizing the vegetation properties are inherently advantageous. The principal disadvantage of

these methods is the difficulty of measuring or estimating the vegetation properties of concern.

While the handbook methods are convenient and offer the advantage of implicitly compositing channel resistance, they offer little utility in cases where vegetation is present or where floodplain flows are anticipated. Only about a dozen such circumstances are addressed in the collective works of the handbooks cited. The disadvantages cited above for direct measurement apply to the handbook methods as well. Additionally, selection of a resistance value based upon photographic comparisons can actually be misleading, resulting in greater errors than simply using qualitative channel and floodplain descriptions (Fischenich and Abt 1995).

The effective area techniques have a sound theoretical basis. However, formulation of methods based upon this concept is in its infancy. Dimensional and theoretical analyses are required to define the significant parameters, and substantial work will be necessary to develop the empirical coefficients that are required to apply these concepts to the solution of actual problems.

The meteorological approaches, though developed for evaluating wind rather than water velocity profiles, may be a useful approach for evaluating resistance in waterways. The concepts have a sound theoretical basis. Additional work is necessary to modify the techniques, develop coefficients, and verify the validity of the approach for use in floodways.

3 Evaluation of Techniques

Analysis

Nineteen channel reaches with measured *n* values were used to assess the prediction methods described above. Four of the reaches were excavated canals, one was a laboratory flume, and the remainder were natural channels. Eight of the natural channels were evaluated for a discharge contained within the banks and seven for overbank flows. Three general cases of vegetal retardance were represented: (a) dense vegetation on the streambanks; (b) submerged or partially submerged aquatic vegetation; and (c) dense vegetation on the floodplains. The effective area techniques and atmospheric sciences techniques were not evaluated because they have not been fully developed for riverine application.

Measured *n* values and those predicted using each of the methods are presented in Table 6. The table also presents the mean of the predicted *n* values and summary statistics for each method. Blanks in the table signify that an *n* value could not be estimated because (a) the method was not applicable to the given conditions, (b) requisite data were not available for the site and could not be estimated, or (c) the magnitude of one or more parameters was well outside the range of data upon which the specified technique was developed. Some liberty was exercised in the use of the methods so that a representation of actual application could be obtained. For example, the *n*-VR/*MEI* method was applied at three sites despite the fact that all had slopes less than the 3-percent minimum used in the formulation of the method. The assumptions made in the application of the method are discussed in the following sections.

Figure 4 presents plots of the predicted *n* values against measured *n* values for each technique. In general, prediction error increased with increasing degree of resistance. Chow's method tended to underpredict resistance at higher values, as did Cowan's. The handbook methods of Barnes and of Hicks and Mason as well as the *n*-VR/*MEI* method displayed no trends relative to overpredicting or underpredicting actual *n* values for the ranges in which they were applicable. The method of Petryk and Bosmajian tended to overpredict resistance and had the greatest degree of variation.

Table 6
Measured and Computed *n* Values and Statistical Summary

Stream/Location	Measured *n*-Value	Predicted *n*-Value						Mean *n*-Value
		Chow	H&M	Barnes	Cowan	P&B	*n*-VR	
Tug Fork River, WV (Bankfull)	0.050	0.040	0.046	0.046	0.054			0.047
Tug Fork River, WV (Overbank)	0.074	0.080		0.080	0.111	0.200		0.109
Pearl River, LA (Marshy Reach)	0.065	0.035			0.074		0.065	0.060
Pearl River, LA (Wooded Reach)	0.095	0.100			0.109	0.190		0.124
Chisolm Creek, near Park City, KS	0.056	0.035	0.032	0.026	0.043			0.038
Hanging Moss Creek, near Jackson, MS	0.074	0.100	0.066	0.070	0.077	0.130		0.086
Gila River, near Yuma, AZ	0.082	0.078	0.046	0.049	0.069	0.097		0.070
Cypress Creek, near Downsville, LA	0.100	0.100			0.105	0.085		0.098
Fall River, near Estes Park, CO	0.110	0.050	0.088	0.065	0.093	0.275		0.114
River Yare, near Norwich, Norfolk	0.150	0.100	0.140		0.117		0.080	0.117
Thompson Creek, near Clara, MS	0.200	0.120			0.155	0.151		0.157
River Bain, U.K.	0.214	0.035			0.095			0.115
Don River, near Toronto, Canada	0.225	0.150			0.150			0.175
River Ebble, U.K.	0.326	0.100	0.120		0.138			0.171
Naanai Canal, Egypt	0.040	0.050	0.060		0.057			0.052
Port-Said Canal, Egypt	0.074	0.080	0.060		0.057			0.068
Kaskaskia Channel, near Bondville, IL	0.080	0.070	0.041	0.045	0.089	0.057		0.064
Two-Mile Sluice, near Sadorus, IL	0.120	0.070	0.066	0.070	0.093	0.065		0.081
Flume w/Bulrush	0.329	0.150			0.112	0.398	0.350	0.268
Number of Points	19	19	11	8	19	10	3	19
Correlation Coefficient		0.614	0.733	0.536	0.718	0.705	0.963	0.897
Standard Error		0.072	0.057	0.022	0.063	0.060	0.052	0.040
Mean Percent Error		32.3	32.6	30.2	26.7	64.9	17.7	22.5

This analysis has provided insight into the applicability and accuracy of the various methods for predicting resistance values for typical field applications. Of particular note is the limited number of cases for which many of the methods could be used. While estimates could be made for all 19 reaches using the methods by Chow and Cowan, the others proved less applicable. Reasonable matches in Hicks and Mason's handbook could be found for 11 cases, and Barnes' handbook could be used for eight. Measurements of vegetation density were available for four sites, but were estimated for an additional six, allowing the method by Petryk and Bosmajian to be used on 10 reaches. The *n-VR/MEI*

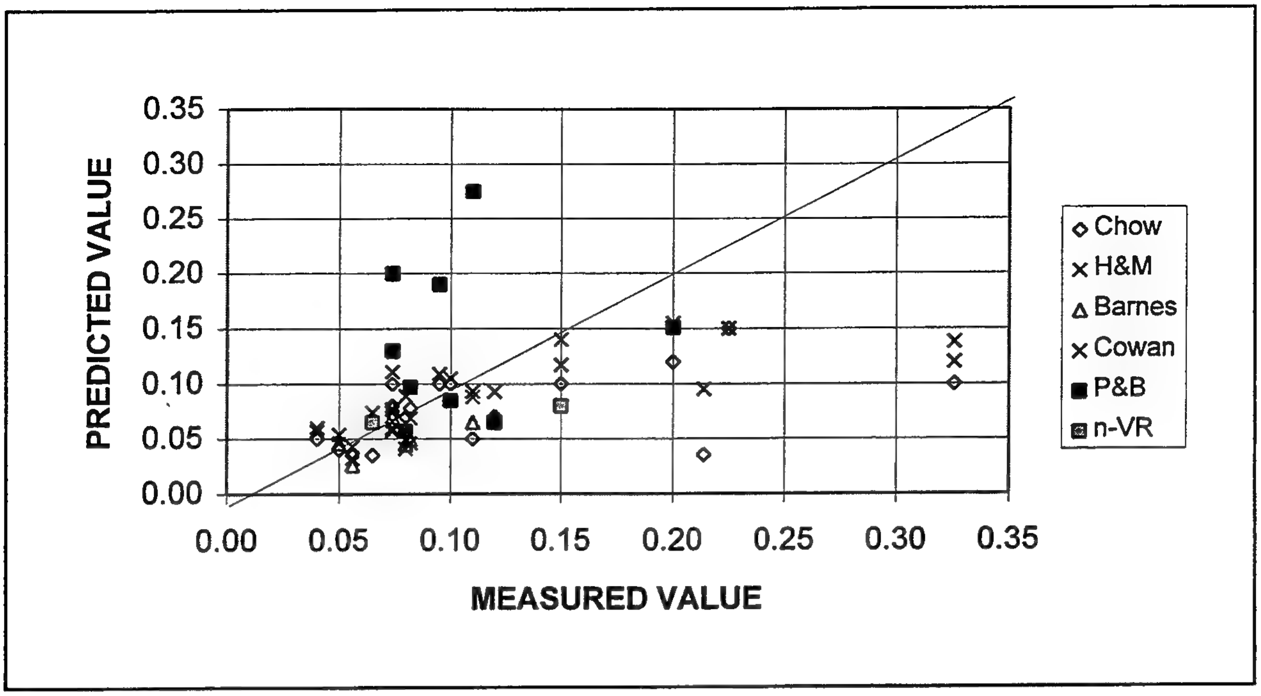

Figure 4. Plots of predicted *n* values versus measured *n* values for each technique

method was applicable to only 3 of the 19 cases investigated and, as discussed, these were somewhat outside the range of data for which the method was developed.

Correlation coefficients for the six methods ranged from 0.536 to 0.963 and averaged 0.712, suggesting that the predicted *n* values may not be a good representation of the actual (measured) *n* values. Standard errors ranging from 0.022 to 0.072 and mean absolute percent errors from 17.7 percent to 64.9 percent support the hypothesis that the methods are poor predictors of *n* values. While the statistical analyses of the results provided some indication of the relative effectiveness of the six procedures used, this exercise yielded many other lessons that bear upon the application of existing techniques for estimating *n* values in vegetated floodways.

Discussion of Results

The concept of assigning a Manning's *n* value to a densely vegetated floodway is illogical, yet there are few alternatives when estimates of a river stage coincident with a design discharge are required. The Manning Monomial Equation (Manning 1891) was developed under the conditions of steady, uniform, turbulent flow. Though approximately steady over a short time step, all of the reaches in this evaluation and in most field applications experience unsteady, nonuniform flow. Additionally, when the density of vegetation is very high and the bed slope is mild, the flow may be transitional rather than fully developed.

The estimates presented in Table 5 were derived by a single investigator. Since each of the methods require some degree of interpretation or the estimation of parameters, other investigators attempting to reproduce the results of this evaluation would likely arrive at different n values. No attempt was made to quantify estimation bias or the variability of estimations by different individuals. The U.S. Army Engineer Hydrologic Engineering Center (HEC 1986) attempted to quantify this variability by having 72 hydraulic and hydrologic engineers estimate the Manning roughness coefficient for 10 streams. For the Gala River, one of the study sites in this analysis, the mean of the estimates by participants was 0.062 and the standard deviation was 0.022. Thus, significant variability can be expected among investigators.

Many investigators hold to the belief that roughness coefficients, such as Manning's n, are constant for a channel reach. This is simply not the case. Both spatial and temporal variability of Manning's n values have been well documented. The variability is particularly pronounced in vegetated channels, as shown by Table 7. Evidence of the temporal variability of roughness coefficients is highlighted by Powell (1978) in which he indicated average seasonal variability on the River Bain between 0.025 and 0.370 (Figure 5), and daily variability averaging 0.116 over a 20-day period in July 1973.

Table 7
Stochastic Characteristics of *n* for Vegetated Channels

		Descriptive Statistics				
Source	**Description**	**No. Obs.**	**Min**	**Mean**	**Max**	**CV**
Bakry, Gates, and Khattab (1992)	28 Canals w/Bank Veg. (space-time)	280	0.011	0.032	0.083	0.40
Bakry, Gates, and Khattab (1992)	9 canals w/Aquatic Veg. (space-time)	156	0.020	0.051	0.183	0.40
Powell (1978)	River Bain Aquatic Veg. (periodic '67-'71)	260	0.020	n/a	0.690	n/a
Powell (1978)	River Bain Aquatic Veg. (15-min intervals '71-'77)	Approx. 120,000	0.020	n/a	4.480	n/a
Watts and Watts (1990)	River Yare Aquatic Veg. (seasonal variability)	9	0.015	0.094	0.160	0.56
Watson (1987)	9 Sites, River Ebble (space-time)	Approx. 20	0.009	n/a	0.412	n/a
Wilson (1973)	Hanging Moss Creek, MS (space-time)	14	0.020	0.045	0.074	0.39

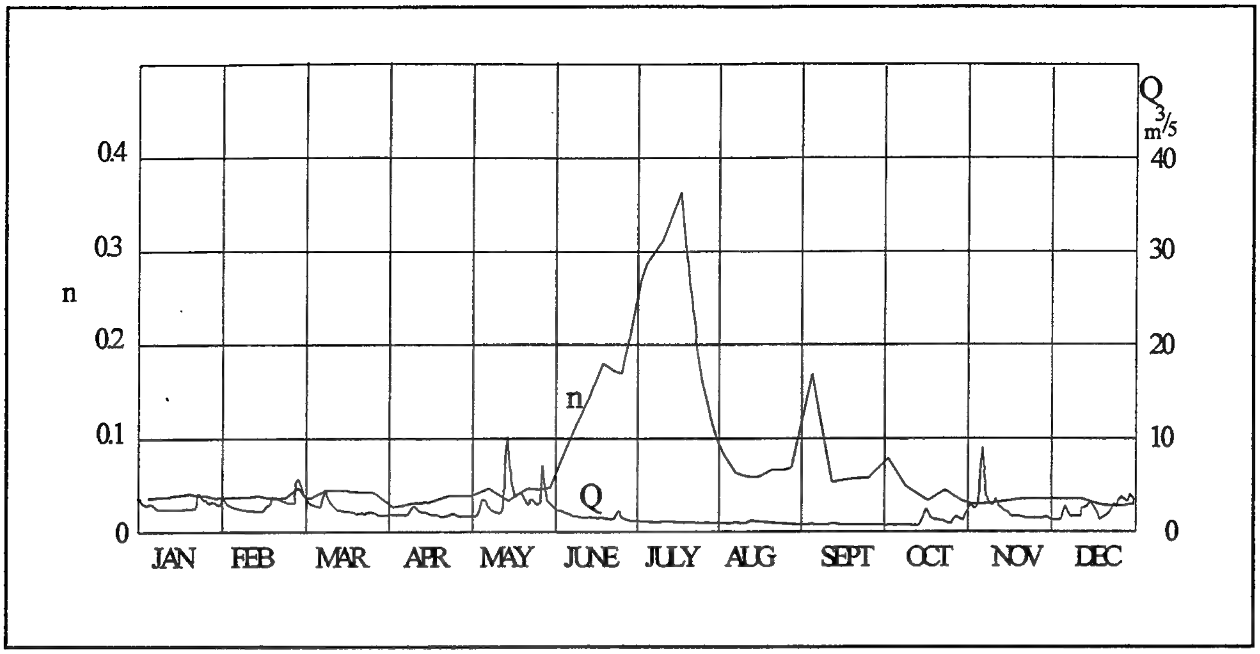

Figure 5. Periodic values of *n* for the River Bain, 1967 (after Powell 1978)

Though Chow's handbook tables and Cowan's procedure rely upon interpretation of qualitative descriptions for the assignment of resistance values, they were applicable to every site and had prediction errors of the same order as the other techniques. However, these methods clearly underpredict resistance for *n* values in excess of 0.1. They are probably best suited to conditions in which vegetation is not present or is confined to the banks. When flows in densely vegetated floodplains are anticipated, alternative procedures should be used or the predicted values should be adjusted. Revision of Chow's and Cowan's tables to include more descriptive information for densely vegetated floodways may improve the predictive accuracy of these techniques.

Handbook methods relying upon pictorial descriptions of channels are convenient and offer the advantage of implicitly composting channel resistance, but they offer little utility in cases where vegetation is present or where floodplain flows are anticipated. Only a dozen such circumstances are addressed in the collective works of the handbooks cited. The authors found that reliance upon these handbooks can actually be misleading, resulting in greater errors than simply using qualitative channel and floodplain descriptions. In many cases, more than one channel in the references appeared to have characteristics similar to those for the channel being investigated, and the reference channels had substantially different resistance values.

Petryk and Bosmajian's vegetation density technique is hindered by the additional data requirements, and its only advantage over Cowan's method is in its application to densely wood floodplains. The method greatly overpredicts resistance for most cases. This may be attributed to the application of Arcement and Schnieder's (1989) procedure for estimating drag coefficients, which yields

values between 6 and 14. More realistic drag coefficients for flows in the turbulent range of most streams would be on the order of 0.5 to 1.5. Despite these limitations, this method yielded the best estimates for wooded floodplains with little undergrowth, and it is recommended that its use be limited to these conditions.

The *n*-VR method is limited by the range of slopes over which data were collected (greater than 3 percent), the types of vegetation used (grasses only), and difficulties in selecting an appropriate curve. The method yielded good estimates of resistance for the three cases in which it was applicable, although the channel slopes were less than 3 percent. The *MEI* analysis proposed by Kouwen et al. explicitly includes the geometric and physical properties of the vegetation in the analysis of flow resistance. Though not substantially verified, this method appears to provide a sound process for the assessment of resistance due to some types of vegetation.

4 Resistance in Composite and Compound Channels

Background

Floodways with vegetation are seldom uniform in vegetation type and density or in cross-sectional shape. Rather, channels proper are typically devoid of vegetation, while their banks and floodplains are vegetated to varying degrees. This nonuniformity in vegetation distribution and channel geometry causes additional momentum losses and raises interesting questions about how best to account for these losses. Several techniques have been developed for dealing with the issue in one-dimensional hydraulic analyses.

A composite channel has a wall roughness that changes along the wetted perimeter of the cross section. A compound channel has a cross section that consists of a combination of subsections of different geometric shapes. Any channel with a defined high- and/or low-flow channel or one that also has flow in the floodplain is considered a compound channel. Virtually all alluvial channels are composite channels, and most are compound channels. Therefore, a compound channel is conventionally regarded as a channel whose cross section not only consists of subsections of different geometric shapes but also has different boundary roughness (composite channel).

Figure 6 is a typical cross section of many flood control channels with offset levees. Between the levees are several "zones" with different roughness elements and geometric shapes. For example, Panels 2, 3, and 4 are all dominated by vegetative roughness elements, each with widely varying resistance values. Whereas resistance in Panels 3 and 4 is primarily the result of form loss due to drag around woody vegetation, resistance in Panel 2 is primarily caused by friction of the flow over herbaceous vegetation. In Panel 5, both grain resistance from the sediment and form resistance from geometric variations (such as bed forms) may be present.

The geometric changes between Panels a, b, c, d, and e can significantly influence the hydraulic parameters of the flow. In a compound channel, shear stresses develop when flow in a main channel is moving much more rapidly

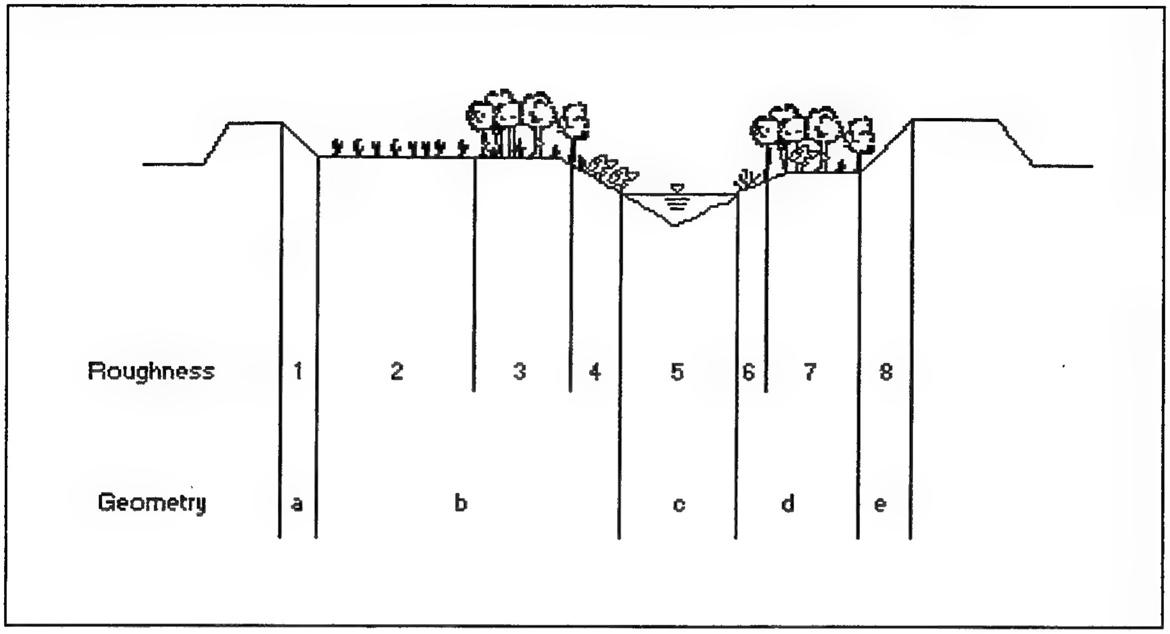

Figure 6. Typical cross section of a flood control channel

than flow on an adjoining floodplain. The difference in flow velocities results in a transfer of momentum from the main channel to the floodplain. In addition, the longitudinal shear stresses cause a loss of energy that reduces the overall compound channel capacity.

The majority of the formulas used for one-dimensional flow analysis were derived for steady uniform flow with the assumption that the momentum resistance slope or the energy resistance slopes of the different subareas of the cross section are the same and equal to that of the entire cross section. In reality, this is not the case because momentum and energy exchanges can and do occur through the internal imaginary boundaries between the panels. In the deeper and faster flowing central region of the channel (Panel c), the momentum and energy losses are higher than those at the shallower and slower side regions of the cross section (Panels b and d). Momentum and energy are continuously transferred from panel to panel. Even for steady uniform flow in a prismatic channel, the momentum or energy slopes are different from subarea to subarea, although the water surface slopes are equal.

The result of the momentum and energy transfer is the loss of conveyance for the channel. Sellin (1964) reported that the shape of a compound channel resulted in a reduction in the conveyance capacity because of the interaction of the main channel and overbank areas. He found that the amount of discharge that could be carried in a channel alone added to what could be carried in the floodplains alone exceeded the discharge that could be carried in the compound channel composed of both main channel and floodplains. The calculated value of Manning's n for his laboratory studies shifted from approximately 0.0088 to a maximum of 0.0100 as the flow reached the top of the main channel and spread into the overbanks, up to a point 0.2 ft deep in the overbank region. Then the

calculated n value decreased as the overbank flow depth continued to rise. Estimates of n that did not account for this interaction between the channel and overbank resulted in an overprediction of the capacity of the compound channel.

Composite channels and compound channels are treated similarly in open-channel hydraulics. The emphasis of the former is on the nonhomogeneous wall roughness, whereas the emphasis of the latter is on both the wall roughness and the geometry. The nondimensional cross-sectional geometry parameter usually consists of more subparameters for compound channels than for composite channels. For a very wide channel, the flow in a subsection is mainly influenced by the flow in the neighboring subsections and practically unaffected by the flow in the far away subsection.

Predictors

The calculations that transform the complex geometry and roughness into representative, one-dimensional hydraulic parameters for flow depth calculations are called Compositing Hydraulic Parameters. That is, in a complex cross section, the composite hydraulic radius includes, in addition to the usual geometric element property, the variation of both depth and n values. There are several methods in the literature for compositing (Yen 1992 and Thomas 1994 [1]).

Interest in compound channel resistance coefficients was promoted not to a small degree by the use of simple one-dimensional computation for backwater curves. For use in one-dimensional model calculations, the resistance of both composite and compound channels is customarily expressed in terms of Manning's n. Most of the compositing formulas listed in the references cited above are commonly considered by engineers to also be applicable to compound channels. However, because of the large depth difference in different subsections, and hence difference in local relative roughness, Reynolds and Froude numbers, the flow geometry interaction is more predominant in compound channels than in composite channels. Therefore, those equations that involve flow parameters R or A are more applicable to compound channels than those not involving these parameters.

The "Alpha Method," based on the Chezy equation, was selected as the default for SAM, the U.S. Army Corps of Engineers' hydraulic design package for channels. The Alpha Method is developed and described in EM 1110-2-1601 (USACE 1991). In the Alpha Method, the cross section is partitioned into panels between coordinate points, and all panels are assumed to be vertical. The cross section is not subdivided between channel and overbanks for this calculation.

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[1] Thomas, W. A. (1994). "Methods for predicting n-values for the Manning equation," Unpublished draft technical report, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Calculations always begin at the first panel in the cross section, and the geometric properties are calculated and saved for each wet panel across the section. The hydraulic radius and Chezy C are then calculated and the compositing parameters summed. Computations move panel by panel to the end of the cross section. The alpha method fails when there is a vertical wall.

Other methods are provided as options in SAM: the equal velocity method that is sometimes called the Horton or the Einstein method after the developers, the Los Angeles District method, the Colbatch method, and the sum of forces method (Thomas 1994).

The equal velocity method, proposed by Horton, and independently by Einstein (Chow 1959), is one that prevents division by zero. Because only wetted perimeter, and not hydraulic radius, appears in this equation, it is always well behaved.

$$n_c = \frac{(p_1 n_1^{1.5} + p_2 n_2^{1.5} + \dots + p_n n_n^{1.5})^{0.67}}{p^{0.67}} \tag{27}$$

where

n_c = composite n value for section

p = total wetted perimeter in cross section

n = last panel in cross section

p_n = wetted perimeter in wet panel n

n_n = n value in wet panel n

The equations for the Los Angeles District and Colbatch methods are listed below. They require assumptions on how the subareas are divided to calculate the area and wetted perimeter. Figure 7 is a definition sketch for the subarea divisions for these methods.

Los Angeles District:

$$n_c = \frac{(a_1 n_1 + a_2 n_2 + \dots + a_n n_n)}{A} \tag{28}$$

Colbatch:

$$n_c = \frac{(a_1 n_1^{1.5} + a_2 n_2^{1.5} + \dots + a_n n_n^{1.5})^{0.67}}{A^{0.67}} \tag{29}$$

where

A = total area in cross section

a_n = area associated with panel n, see definitions in Figure 6

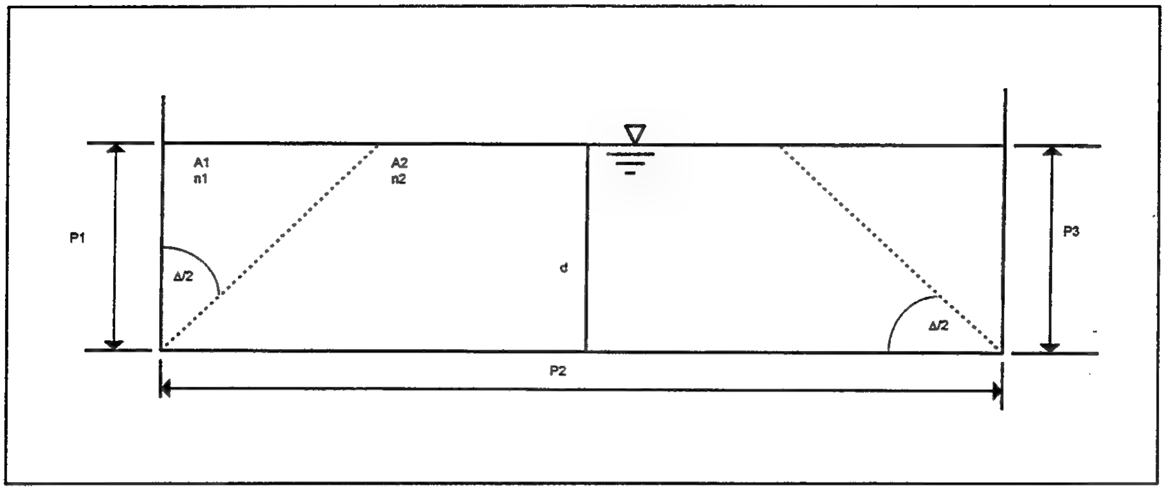

Figure 7. Definition sketch for Los Angeles District and Colbatch methods

where

$P_1 = P_3$ = side length or wetted perimeter for areas A_1 and A_3

P_2 = bottom width or wetted perimeter for area A_2

d = depth of flow

$A_1 = A_3$ = area bounded by side and bisector of D (90 deg) angle

A_2 = area bounded by bottom and bisectors of D angles

n_1, n_2, n_3 = Manning's n value for respective wetted perimeters

Δ = angle between channel side and bottom (90 deg)

As a result of several experiments, Cox (1973) concluded that Horton's method was not as accurate as the Los Angeles District method or the Colbatch method. Based on one of Cox's figures, the Horton method gave a composite n value as much as 8 percent higher than measured for the combination of rough walls and a smooth bed. One test, a combination of smooth walls and a rough bed, gave an effective n value about 4 percent lower than measured.

Horton's method is retained in SAM because of its simplicity. It is adequate for the simple cross-section shapes, and it is programmable for the complex

cross-section shapes. The other methods that Cox tested would be very difficult to program for automatic computations in complex cross sections.

The Sum of Forces Method, proposed by Pavlovskii, Muhlhofer, Einstein, and Banks (Chow 1959), is based on the hypothesis that the total force resisting the flow is equal to the sum of the forces resisting the flow in each panel. The resulting composite *n* value is equivalent to that for the Los Angeles Method (Equation 27).

The traditional approach to compositing in HEC-2 and HEC-6 is by the conveyance method. The conceptual basis for this approach is that the sum of the subarea discharges equals the total discharge. This method of compositing was developed by Lotter in 1933 (Yen 1992):

$$n_c = \frac{PR^{1.67}}{\sum P_i R_i^{1.67} / n_i} \tag{30}$$

Researchers have generally assumed that Lotter's equation is more adaptable than other equations to compound channels and to composite channels. The equations used for HEC-2 and HEC-6 differ slightly from Lotter's but are based upon the same principle.

Motayed and Krishnamurthy (1980) compared five different approaches for calculating the composite roughness of a channel using U.S. Geological Survey (USGS) cross-sectional data from 36 different streams in Maryland, Georgia, Pennsylvania, and Oregon. They compared the results from four of the equations to the results of an analysis that developed a calculated *n* value using detailed velocity data from each cross section, an assumption of logarithmic velocity distributions in each subarea, and application of the Manning-Strickler formula. The results of this equation were assumed to be the "true" *n* value against which the results of the other methods were compared.

The equations they evaluated included estimates of composite *n* value assuming that (a) mean velocity is uniform across the cross section, relating subarea wetted perimeters and *n* values to the total wetter perimeter and *n*-value (Horton's formula); (b) total resistance equal to the sum of resistances in each subarea and a hydraulic radius for each subarea equal to the hydraulic radius for the entire cross section, again relating subarea *n* values and wetted perimeter values for each subarea to a composite *n* value (sum of forces method); (c) total discharge is equal to the sum of the subarea discharges, relating total and subarea wetted perimeter and hydraulic radius and subarea *n* values to a composite *n* value (Lotter's formula); and (d) a logarithmic velocity distribution, relating subarea and total wetted perimeter, depth, and subarea *n* value to a composite *n* value.

Lotter's equation was shown to have the least scatter in *n* values compared to the assumed "true" value. The authors acknowledge that the assumption of logarithmic velocity distributions in high-flow stages may not be valid.

Although the available experimental data suggest that calculation of discharge through application of a technique relying on division into subareas is a crude tool at best, it is a widely recognized and easily applied approach that requires no additional data collection. It also offers a technique for indirectly calculating values of channel and overbank roughness if stage, discharge, and geometry data are available. Lastly, this approach is the most likely to be employed for estimating discharge using channel and overbank roughness estimates that are developed by any of the available methods.

The method used to divide the cross section into subareas affects the determination of A_i, P_i, and R_i and hence, the compound channel resistance coefficient n. Different suggestions have been made on how to divide the cross section into subsections. They can be classified into five groups: (a) dividing the floodplains from the main channel by using a vertical line extending from each of the break points between the main channel and floodplains, (b) dividing the cross section by a horizontal or almost horizontal line joining the two break points at bankfull stage, resulting in upper and lower channel sections, (c) dividing the cross section by using the bisect line of the angle at the break point; for a narrow main channel if the bisect lines meet below the water surface, an adjustment is made by extending this meeting point upward to the water surface, (d) a slight variation of the above is dividing by a straight line from the water surface at the middle of the main channel to the break point, and (e) dividing by using a diagonal straight line or curve, usually with the intention to match the dividing line as close to the zero-shear surface as possible. Figure 8, adapted from Andrews (1993), can be used to envision these divisions.

In Figure 8,

H: 1-1

D: 1-2

V: 1-3

H_i, D_i, V_i = Division line is included in the wetted perimeter of the main channel

H_e, D_e, V_e = Division line is excluded from the wetted perimeter

Following the convention of Figure 8, Andrews (1993) provided a discussion of the literature in terms of the applicability of the various methods of division. As can be seen from the following discussion, there is a diverse body of opinion as to which method is best under various circumstances. In general, this author has concluded that for floodplain flow depths less than one-third the main channel flow depth, the H_i method is preferable. For floodplain flow depths greater than or equal to one-third the main channel flow depth, the D_e method is preferred.

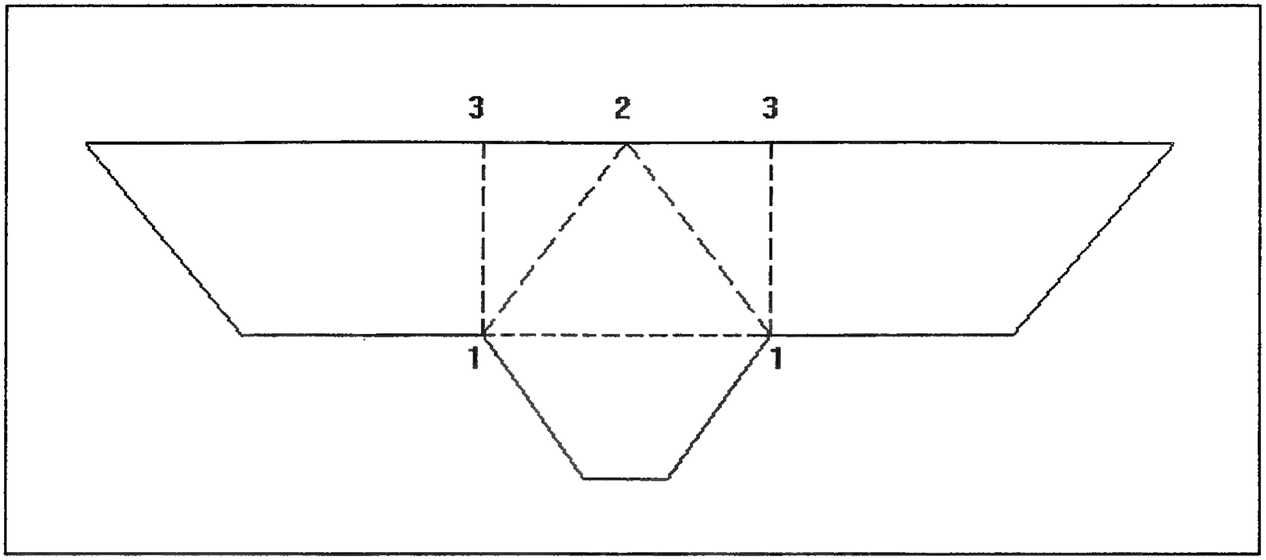

Figure 8. Definition sketch for channel divisions

Wormleaton, Allen, and Hadjipanos (1982) used a symmetrical compound section to examine the relative discharge capacity of the main channel and floodplains. Their experimental channel had floodplains with four different *n* values varying from 0.11 to 0.21 on the floodplains and an *n* value of 0.10 in the main channel. They developed expressions for the horizontal and vertical apparent shear stresses at the boundaries between the main channel and floodplain flow. They suggest that if the apparent shear stresses could be calculated, they could be used to suggest the most appropriate technique for dividing the channel into subareas for discharge estimation. Where these apparent shear stresses are small, they might be ignored, or if they are similar to boundary shear stresses, they may be used to locate a subarea boundary. Alternatively, some other means of accounting for their presence could be identified.

Using data from part of their own work and three other studies of both symmetrical and asymmetrical floodplains, they used regression analysis to develop an equation for apparent shear stress and compared it with a limited number of their own results for validation. The equation appeared to represent observed values quite well, though it was based on only 55 sets of results. Their analysis showed a strong correlation between apparent shear stress acting on vertical interface planes and the differences in velocity between subsections, the ratio of depths of flow in the floodplain and main channel, and the ratio of the width of flows on the floodplain and in the main channel.

They found that apparent shear stresses across horizontal and diagonal planes were much smaller than those across vertical interfaces. For deeper flow depths in the case of horizontal, and to some extent diagonal, interfaces, the momentum transfer across these interfaces was found to be from the floodplain to the main channel. At lower depths, the transfer was from the main channel to the floodplain.

The authors examined three different methods of dividing the compound channel into subareas. They analyzed vertical division methods, in which vertical boundaries are envisioned extending from the edge of the main channel bank to define the main channel and floodplain edges. They also examined the case of horizontal subdivision, in which horizontal boundaries are assumed to extend across the main channel, at the elevation of the floodplain. Lastly, they looked at diagonal division planes, extending from the edge of the main channel bank to the center of the main channel at the water surface. Two cases were examined for each method, one in which the subarea boundaries were included in the wetted perimeter of the main channel only, and one in which they were excluded. Each method of analysis was applied to each of the four floodplain *n* value conditions at several different depths of flow.

The authors found that the ratio of calculated discharges to observed discharges tended to increase with increases in roughness for all calculation methods. Method V_e was found to consistently overestimate discharges, even at higher depths. Methods V_i and D_e yielded similar results, though the latter was more accurate. For low-flow depths and higher roughness, D_e still substantially overestimates discharge. The ratio of calculated-to-observed discharge generated by H_e tended to be lower, and method H_e was found to underestimate discharge at higher flow depths. Methods H_i and D_i yielded similar results, though flows tended to be underestimated except for lower flow depths on channels with rougher floodplains. All of the methods underestimated the total discharge in some cases, but never more than about 20 percent. Overestimates of discharge of up to 80 percent also occurred, particularly with rougher floodplains. To develop a conservatively low estimate of the discharge capacity, the authors concluded that use of method H_i or D_i should be considered. In general, the *H* and *D* methods yield better results than the *V* methods overall.

Comparison of the ratio of apparent shear stress to theoretical average shear stress around the boundary of the main channel subdivision (including vertical or other interfaces) was carried out for each case. The results suggest that calculation of this value is in fact a good indicator of whether the subarea interface should be included in the wetted perimeter for the most accurate estimate of channel discharge.

Prinos and Townsend (1984) reviewed a variety of calculation methods and proposed a new approach. Four of the seven methods presented relied on division into subareas. Some of the methods had been proposed by earlier researchers, and experimental results from earlier work and Prinos and Townsend's own flume studies were reviewed. One method reviewed involved calculation of a composite *n* value using each of the common methods of calculating a composite *n*. They showed that these methods resulted in consistent underestimation of the discharge by up to 37 percent for their experimental data due to faulty assumptions in the composite *n* value equations. An approach using a vertical plane to subdivide the main channel and floodplain, with and without inclusion in the main channel wetted perimeter (V_i and V_e respectively), was also tested. The former overestimated discharge for low floodplain depths, presumably because the method assumes equal apparent shear

stress and boundary shear stress in the main channel. The latter overestimated total discharge because it ignores the effect of the apparent shear force on the imaginary boundary interface plane. The method of using a horizontal interface with no inclusion of the interface in the wetted perimeter (method H_e) was also tested and found to give somewhat better results, though discharge was usually still overestimated at low floodplain depths.

Wormleaton and Hadjipanos (1985) conducted further research using the same experimental channel and *n* values that they used in their 1982 work. Their goal in this effort was to determine the method that most accurately estimated the discharge in each of the channel subareas, rather than simply the total channel discharge. They used four different methods of dividing the channel cross section into subareas and measured the percentage error in discharge estimates that resulted. They analyzed V_i, V_e, H_i, and H_e, as defined above. Each method of analysis was applied to each of the four floodplain *n* value conditions at three different depths of flow.

The authors found that even the method that most successfully estimated total discharge in these 12 situations had as much as a 22-percent error and that none of them satisfactorily calculated flow between the main channel and the floodplains. The vertical methods tended to overestimate main channel discharge and underestimate floodplain discharge. Method H_e tended to give reasonable main channel discharge estimates when the floodplains were smooth, but overestimated discharge for rougher floodplains. Method H_i gave better main channel estimates for shallow flow across rough floodplains, but was less accurate when floodplains were smoother or flow was deeper. At higher floodplain depths, the vertical methods gave better total discharge estimates, but the horizontal methods did a better job of allocating discharge between subareas.

The Hydraulic Research Institute (1988) conducted research on the accuracy of discharge calculations using data from water authorities in England and Wales for rivers with overbank flow. Their work suggested that for straight main channels with parallel berms, method D_e gave the best results, though V_i was also fairly accurate. Discharge estimates were improved when floodplain flow was ignored altogether for cases where $y_r < 0.1$. They also recommend the use of H_i, with the division line added to both subareas, for cases with a meandering main channel, provided $y_r > 0.3$.

Ramsbottom (1989) reviewed data that had been collected at 15 different rivers in the United Kingdom for which overbank flow data were being collected. The investigator used the division into subareas method to estimate discharge at nine sites, basing the *n* values for the main channel on the calculated bankfull discharge n and the overbank *n* on estimates based on traditional estimation techniques. The V_i, V_e, D_i, H_e, and single channel methods were used to estimate discharge based on these *n*-value assumptions. The author concluded that most methods produced predictions within 10 percent, with the exception of H_e and the single channel method, which underpredicted the discharge. Methods V_i and D_e were identified as good if reasonable roughness estimates could be made. The

results indicated that the predictions using method D_e were somewhat more conservative than those produced using method V_i.

More recently, Wormleaton and Merrett (1990) used another experimental compound channel to explore this issue further. The channel used in this study was a symmetrical channel with sloping main channel and floodplain sides except for the case of the widest floodplain, which had vertical sides. One of the five configurations included a roughened floodplain with a floodplain to main channel bottom width ratio of three. The other cases used a smooth channel with floodplain to main channel bottom width ratios of 5.47, 3, 1, and 0 (the latter representing a trapezoidal channel without floodplains).

Wormleaton and Merrett found that the discharge for the roughened floodplain case was even less than the trapezoidal channel case at up to a floodplain to main channel depth ratio of 0.5. This clearly indicated the retarding effect of the roughened floodplain on the main channel flow. They found a Manning's n value for the main channel of 0.01, based on the trapezoidal channel. To estimate the n value that would be represented by the roughened floodplain case, they used an assumption of two-dimensional flow and measured flow characteristics outside the region assumed affected by the main channel-floodplain interface or side shear. This effort yielded an estimate of n values for the floodplain that ranged from 0.0156 to 0.451 for relative floodplain to total flow depths of 0.04 to 0.5.

Three methods were examined for estimating discharge using the subarea technique: V_i, D_e, and H_e. For the total channel discharge, the authors found that method V_i gave the highest value over most of the depth range and method H_e the lowest. Except for the widest floodplain case using method H_e, the error in estimated total discharge decreases for all of the approaches with increased floodplain depths. Method D_e was found to perform best overall, especially for narrower floodplains.

Errors in estimates for main channel discharge were much higher than those for the total discharge. These errors did not decrease to the same extent at higher floodplain depths as the total discharge estimates did. The same order of discharge estimates existed for main channel flow estimates, with V_i yielding the highest values and H_e the lowest. Methods V_i and D_e appear to overestimate main channel discharge under most conditions, V_i to a greater extent. Method H_e yielded underestimates for main channel discharge in most cases, especially at higher floodplain depths.

Other Literature

Almost all of the investigations of compound channels have been for steady uniform flow. Most of them assume the logarithmic velocity distribution and the Colebrook-White or similar resistance coefficient formulas hold, and the resistance coefficients remain constant for a given type of surface, unchanged by either the geometry of the channel or the depth of the flow. A good review of

compound channel problems was given in the paper by Williams and Julien in Yen (1992). Andrews (1993) provided an excellent summary of the literature on composite and compound channels. Following is a summary of these reviews:

Williams and Julien in Yen (1992) found that as the ratio of the floodplain to the channel widths (aspect ratio) increased, the apparent roughness decreased, leading to the conclusion that total discharge is overestimated or underestimated, depending on the calculation method used. This roughness decrease was most evident for low ratios of total flow depth to channel depth (total depth less than floodplain depth), Y/D, but was negligible for Y/D greater than 1.4. In the range $1 < Y/D < 1.4$, traditional single or separate channels methods should not be used, but for Y/D greater than 1.4, no corrections to the methods are required. They found that the longitudinal slope does not have much influence on the interaction.

Work reported in Knight and Demetriou (1983) was based on measurement of shear force distributions in a smooth-surfaced flume constructed with an adjustable adjoining floodplain on each side of the main channel. In general, the existence of the adjoining floodplains served to slow the flow in the main channel when depths exceeded the elevation of the floodplains. The apparent shear force was found to vary systematically with the ratio of total width to main channel width and the ratio of floodplain depth to main channel flow depth. They found a peak apparent shear force along a vertical interface at low floodplain depths and high floodplain widths relative to the main channel. The vertical shear force between the main channel and the floodplains was found to always be positive, thus indicating a retarding of flow in the main channel, and the horizontal shear force between the lower main channel and the upper main channel with floodplains became negative at greater flow depths, indicating an acceleration of lower main channel flows. Equations were developed to fit their experimental data that predicted the percentage of flow that was carried in the main channel and the lower main channel for the conditions modeled.

Baird and Ervine (1984) performed 136 tests with an experimental flume, measuring point velocities, boundary shear stress, stage, and discharge for an experimental flume at a range of bed slopes. They found that the presence of overbank flow could increase the Darcy-Weisbach friction factor in the main channel by 0 to 30 percent. Analysis of their results showed a complex relationship between the change in friction factor and relative depths of flow in the main channel and floodplain, velocity differences between the main channel and floodplain, and cross-sectional geometry. They also developed a correlation between the nondimensional apparent shear stress and the relative depths of flow, using both their data and the results of earlier research for smooth asymmetric channels. Lastly, they developed a power correlation between the discharge carried in the main channel section during overbank flow and the ratio of the channel depth to the bankfull depth.

Pasche and Rouve (1985) undertook an analysis of the Darcy-Weisbach friction factor that would be associated with the components of a compound channel to examine the effects of both channel shape and vegetatively roughened

floodplains, especially the interface plane between a main channel and a vegetatively roughened floodplain. The existence of four different areas was hypothesized: (a) floodplain not influenced by main channel, (b) floodplain influenced by channel, (c) main channel influenced by floodplain, and (d) main channel not influenced by floodplain. They reported on the development of an exact equation for the drag coefficient for cylindrical rods used in their analysis. Empirical analysis was used to develop an equation for the velocity ratio, and several simplifying assumptions were made to allow the analysis of the friction factor of the interface assuming one-dimensional flow with measurable data. They found that the value of the interface friction factor was primarily dependent on the ratios of cylinder diameter to vegetation spacing and a term referred to as "cooperating width," or the width across the cross section in which flows are affected by the interface between main channel and floodplain flows.

The authors used an experimental compound channel section with variable floodplain widths and roughness to investigate the theoretical relationships they had developed. They performed regression analyses to determine the empirical constant needed to estimate the friction factor of the main channel boundary and found good agreement between predicted and experimental results. In comparing their results with the results of other experiments under uniformly hydraulically smooth conditions, they found that the slopes of the main channel bank make a substantial difference in the apparent shear stresses between the main channel and the floodplain, with sloping banks resulting in lower apparent shear stresses and thus lower resistance. Good agreement was also found between the equation and observed values of the floodplain friction factor in the region not influenced by main channel flow. They found that the friction factor for the imaginary wall or interface between the floodplain and main channel was not dependent on main channel depth when the ratio of channel to floodplain depth was less than 3.0. They suggested that the influence of main channel depth and therefore the main channel bank on the interface friction factor can be ignored for overbank flow with high floodplain roughness. Multiple regression analysis was used to develop parameters that produced good agreement between calculated and observed values for their equation. They also used data analysis to develop an equation for dimensionless slip velocity based on a dimensionless vegetation parameter. The cooperating width was calculated with another equation based on the balancing of momentum in the portion of the floodplain influenced by main channel flow, and they found that it gave good agreement with observed values.

Tests carried out with various widths of floodplain vegetation showed that the friction factor of the interface varies very little in response to the change in width of the floodplain vegetation, even though they found much higher velocities in the partially vegetated floodplain case. Thus, the investigators concluded that the value of the friction factor for the interface cannot be sufficiently described by the differences in velocity between the floodplain and main channel subareas, and the idea of a cooperating width is supported. The authors tested their equation for interface friction factor using data from the River Ihme and found good agreement between the predicted value and the value calculated based on

an assumption of proportional distribution of resistance between the boundary and the interface.

Myers (1987) showed that for a smooth compound channel, the ratios of channel with floodplain Reynolds number, velocity, and discharge are independent of slope and depend only on the ratio of the depth of flow over the floodplain to the depth of flow in the channel. Unlike Knight and Demetriou, Myers found that the velocities observed on the floodplains actually exceeded the velocities in the main channel at very high ratios of floodplain to channel depth. Of necessity, at very large depths, the velocities in each subarea become equal.

Myers compared the amount of discharge observed in the compound channel with the amount of total discharge that the subareas could carry if isolated from each other and found that for the experimental channel, the carrying capacity of the compound channel was 10 percent greater at large relative flow depths (e.g., ratio of floodplain flow depth to total flow depth of 0.5) than would be predicted by a divided channel method that assumes that the apparent shear stress along the division lines is equal to the boundary shear stress. This is true because main channel flows are relatively accurately predicted under these conditions, but floodplain flows are underpredicted because of the momentum transfer to the floodplain. However, at low-flow depths, overestimation of carrying capacity was up to 10 percent. The overestimation occurred at relative flow depths for the floodplain to channel of up to 0.4. As Myers points out, this overestimation percentage is probably low for natural channels, since it would increase with floodplain width and roughness. The discharge capacity of the floodplains, on the other hand, was underestimated as a result of noninteraction by up to 26 percent, peaking at a relative floodplain to channel depth of 0.4 and remaining consistently at 10 percent or greater for the range of relative depths studied (approximately 0.18 to 0.5).

Myers and Brennan (1990) provide a graph of relative depth (floodplain to main channel) versus the ratio of discharge observed in the main channel of a compound channel to the discharge that occurs in the main channel when separated by a wall from the floodplain. Unlike Myers' similar graph in his 1987 paper described above, this graph shows main channel interacting discharges at up to 10 percent less than noninteracting discharges even at a relative depth of 0.5. Generally, the results obtained from the channel configurations used in this study show greater levels of main channel capacity reduction when acting as part of a compound channel than the results of the experiments conducted earlier at the same relative depths did. The differences in the channel configuration—vertical versus sloping boundaries—could easily explain these differences, because Myers and Brennan present a graph that illustrates the difference that channel configuration makes on these results.

The authors provided a graph of stage versus n value (noting that Manning's equation is only strictly applicable in the rough turbulent zone, but acknowledging the prevalence of its use). This graph shows a substantial decrease in the total n value for the compound section at flow depths just greater

than bankfull. Compound channel n values then rise with depth above bankfull, reaching the expected n value appropriate for a trapezoidal channel case at a relative depth of approximately 0.5. This suggests that for moderate relative depths, the capacity of the total cross section of a compound channel may be substantially underestimated by the assumption that a compound channel's n value remains constant when depths greater than bankfull are reached. Flows in the main channel may be overestimated, using the n value appropriate to the trapezoidal channel; however, floodplain flows would be underestimated. The momentum transfer mechanism is responsible for the lower velocities in the main channel and higher velocities in the floodplains (or higher and lower n values, respectively) than would be expected if each of these subareas were noninteracting.

For the main channel and floodplain, subareas were calculated based on full velocity traverses and main channel n values at flow depths above bankfull for the compound channel. They were shown to be much greater than n values for a trapezoidal shape configuration. Similarly, floodplain n values were generally lower than the n value determined for the trapezoidal shape. At low depths across the floodplain, discharge calculations using separate subareas and the same n value determined for the trapezoidal case led to overestimation of compound channel carrying capacity since the main channel conveys most of the flow at these depths.

Plots of n value and friction factor versus Reynolds number for each configuration are provided, and the compound channels show increasing n-values for increasing Reynolds number. The trapezoidal channel, on the other hand, shows a slightly negative relationship between these variables.

Higginson and Johnston (1992) measured velocities across a constructed, vegetated compound channel section of the River Main in northern Ireland. The terraces are covered with heavy, unmanaged weed growth. Higher velocities were found in the center of the mildly sloped flood terrace than at either the outside terrace edge or the edge next to the main channel. For the channel, n values were calculated as a whole, and they were found to initially drop with increased water depth until flow reached the top of the terrace, and then to rise as the depth of flow on the terrace increased. A maximum measured value of 0.055 was reached at the greatest depth of 1.9 m on the terrace. Using a vertical division method, they calculated a maximum n value for the terrace alone of 0.16.

Laboratory tests on a physical model of the channel showed similar variations of n with depth, with the exception of a fully smooth compound channel. In this case, n values decreased when the water level rose above a minor depth on the terrace. A slight decrease in n value was also observed at large water depths on the fully rough model, but only when a substantial water depth was reached. They were able to calibrate a mathematical turbulence model to fit laboratory measurements of stage versus discharge for both a rough- and smooth-channel condition. Predicted discharges were slightly low at high depths and slightly high at low depths.

5 Conclusions and Recommendations

Many engineers and scientists rely upon experience and judgment, along with a standard reference such as Chow's tables, to select resistance coefficients for hydraulic and channel stability analyses. Presumably, a more quantitative approach would improve accuracy and prove more useful for investigations where dense vegetation is present in the floodway. A predictive method incorporating variables such as flow depth, percentage of the wetted perimeter covered by vegetation, vegetation density, vegetation response to flow, and vegetation alignment would be expected to yield more accurate predictions than methods based upon judgment or the interpretation of qualitative descriptions. No such predictor has been proposed, but the methods evaluated in this investigation include some of these variables.

For the 19 reaches evaluated in this study, the performance of the six methods tested was largely a function of the vegetation characteristics. In cases where the vegetation was limited to the banks or where densities were low, all of the methods performed reasonably well. When the measured n value was less than 0.10, the mean error of estimation for the six methods was +4.1 percent (24.7 percent absolute error). Performance in cases with higher resistance was poor, however; the six methods collectively underpredicting measured resistance by an average of 39.7 percent of the measured value.

Nonuniform velocity distribution and low Reynolds numbers are common in densely vegetated channels and floodplains, making the use of Manning's Equation rather dubious. As a consequence, n values in the range of 0.10 to 0.30 are common and values exceeding 1.0 are possible. Engineers must condition themselves to accept these "high" resistance values, and the existing handbooks and tables should be revised to include conditions under which extreme resistance values can be encountered.

The spatial and temporal variation of Manning's n can be significant in vegetated floodways and canals. Seasonal and even daily fluctuations of an order of magnitude have been shown. Though model calibration and verification are strongly encouraged, the variability of resistance coefficients suggests that the calibration and verification process provides no assurance that extreme

conditions are adequately represented. Thus, the use of stochastic analyses that account for the variability and distributions of resistance coefficients may be the only reasonable approach to hydraulic evaluations of vegetated floodways.

None of the six methods tested proved satisfactory for measured *n* values in excess of 0.10. The benefits of using physically based approaches such as the *n*-VR/*MEI* and Petryk and Bosmajian methods are largely offset by the additional data requirements and uncertainties in coefficient or curve selection. The handbook methods, while offering simplicity, tend to discourage the selection of *n* values in excess of 0.10, and the pictorial handbooks can be quite misleading. In general, it can be concluded that each of the methods offer insight into a probable range of *n* values for cases where the resistance due to vegetation is not extreme, but will grossly underpredict resistance in cases where the vegetation resistance is great.

Resistance prediction becomes more difficult in the case where channel boundaries have differing roughness or the channel has a complex geometric shape. The calculations that transform the complex geometry and roughness into representative, one-dimensional hydraulic parameters for flow depth calculations is called Compositing Hydraulic Parameters. There are several methods in the literature for compositing. Given the diversity of opinion as to which is the most appropriate, much remains to be learned about these techniques and the physical conditions that they attempt to represent.

Additional work on the development of resistance relationships for vegetated floodways should focus on a procedure that minimally exhibits the following characteristics: resistance should be related to readily defined, measurable characteristics of the channel, vegetation, and flow; resistance should be described as a continuous function of the independent variables involved; and the resistance function should be dimensionally homogeneous.

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Healthy riparian vegetation tends to stabilize streambanks, provides shade that prevents excessive water temperature fluctuations, performs a vital role in nutrient cycling and water quality, improves aesthetic and recreational benefits of a site, and is immensely productive as wildlife habitat. Concurrent with these benefits are impacts to the channel and floodway conveyance with subsequent sedimentation and stability impacts. An estimate of the resistance to flow is required to conduct the hydraulic and geomorphic assessments necessary to quantify these impacts.

This report summarizes existing techniques for the prediction of resistance coefficients associated with riparian vegetation. Estimation procedures are grouped into five categories: direct measurement, analytical, handbook, effective area, and meteorological. Advantages and limitations for each of the categories are discussed, and several techniques are evaluated by application to field data for 19 densely vegetated channel reaches.

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